PROCEEDINGS

OF THE

AMERICAN SOCIETY OF CIVIL ENGINEERS

VOL. 67 DECEMBER, 1941 No. 10

TECHNICAL PAPERS

AND

DISCUSSIONS

Published monthly, except July and August, at Prince and Lemon Streets, Lancaster, Pa., by the American Society of Civil Engineers. Editorial and General Offices at 33 West Thirty-ninth Street, New York, N. Y. Reprints from this publication may be made on condition that the full title of Paper, name of Author, page reference, and date of publication by the Society, are given.

Entered as Second-Class Matter, September 23, 1937, at the Post Office at Lancaster, Pa., under the Act of March 3, 1879. Acceptance for mailing at special rate of postage provided for in Section 1103, Act of October 3, 1917, authorized on July 5, 1918.

Subscription (if entered before January 1) $8.00 per annum. Price $1.00 per copy

CURRENT PAPERS AND DISCUSSIONS

		Discussion closes
Reliability of Station-Year Rainfall-Frequency Determinations. *Katherine Clarke-Hafstad*	Nov., 1940	
Discussion	Jan., Feb., Mar., Apr., May, June, 1941	Closed*
Fort Peck Slide. *T. A. Middlebrooks*	Dec., 1940	
Discussion	Mar., Apr., May, Sept., Dec., 1941	Closed*
Rigid Frames Without Diagonals (The Vierendeel Truss). *Louis Baes*	Jan., 1941	
Discussion	June, Sept., 1941	Closed*
Concrete in Sea Water: A Revised Viewpoint Needed. *Homer M. Hadley*	Jan., 1941	
Discussion	Mar., Apr., May, June, Oct., Nov., Dec., 1941	Closed
Value of Public Works. *J. P. Hallihan*	Feb., 1941	
Discussion	Apr., May, June, Sept., Dec., 1941	Closed
On the Method of Complementary Energy. *H. M. Westergaard*	Feb., 1941	
Discussion	Apr., Sept., 1941	Closed*
Crack Prevention Program, Hiwassee Dam. *O. Laurgaard*	Mar., 1941	
Discussion	Sept., Dec., 1941	Closed
Formulas for the Transportation of Bed Load. *H. A. Einstein*	Mar., 1941	
Discussion	June, Sept., Dec., 1941	Closed
Design of Acceleration and Deceleration Lanes. *Adolphus Mitchell*	Mar., 1941	
Discussion	May, June, Sept., Dec., 1941	Closed
Simplified Theory of the Self-Anchored Suspension Bridge. *C. H. Gronquist*	Feb., 1941	
Discussion	May, Sept., 1941	Closed*
Some Economics of Airports. *W. Watters Pagon*	Feb., 1941	
Discussion	Sept., 1941	Closed*
Missouri River Slope and Sediment. *William Whipple, Jr.*	Mar., 1941	
Discussion	Sept., Nov., 1941	Closed*
Analysis of Building Frames with Semi-Rigid Connections. *Bruce Johnston and Edward H Mount*	Mar., 1941	
Discussion	May, June, Sept., Oct., 1941	Closed*
The Suspension Bridge Tower Cantilever Problem. *Blair Birdsall*	Apr., 1941	
Discussion	Sept., 1941	Closed*
Consumptive Use of Water for Agriculture. *Robert L. Lowry, Jr., and Arthur F. Johnson*	Apr., 1941	
Discussion	June, Sept., Oct., 1941	Closed*
Tunnel Construction, Sixth Avenue Subway, New York, N. Y. *Jacob Feld*	Apr., 1941	
Discussion	Nov., 1941	Closed*
Moments in Continuous Rectangular Slabs on Rigid Supports. *L. C. Maugh and C. W. Pan*	May, 1941	
Discussion	June, Sept., 1941	Jan., 1942
Surface Runoff Determination from Rainfall Without Using Coefficients. *W. W. Horner and S. W. Jens*	Apr., 1941	
Discussion	June, Oct., Dec., 1941	Jan., 1942
Operation Experiences, Tygart Reservoir. *Robert M. Morris and Thomas L. Reilly*	Apr., 1941	
Discussion	Oct., 1941	Jan., 1942
The Surveyor and the Law. *A. H. Holt*	May, 1941	
Discussion	Oct., Dec., 1941	Jan., 1942
Compaction of Cohesionless Foundation Soils by Explosives. *A. K. B. Lyman*	May, 1941	
Discussion	Oct., 1941	Jan., 1942
Extensometer Stress Measurements, North Avenue Bridge, Chicago, Ill. *Lawrence T. Smith and Paul Lillard*	May, 1941	
Discussion	June, Oct., 1941	Jan., 1942
Evaluation of Flood Losses and Benefits. *Edgar E. Foster*	May, 1941	
Discussion	Sept., Oct., Dec., 1941	Jan., 1942
Traffic Engineering as Applied to Rural Highways. *Milton Harris*	May, 1941	
Discussion	Sept., Oct., Dec., 1941	Jan., 1942
Pile-Driving Formulas: Progress Report of the Committee on the Bearing Value of Pile Foundations	May, 1941	
Discussion	Sept., Oct., Nov., Dec., 1941	Jan., 1942
Wind Stress Analysis by the *K*-Percentage Method. *F. P. Witmer*	June, 1941	
Discussion	Sept., Oct., Nov., 1941	Jan., 1942
Salts in Irrigation Water. *Raymond A. Hill*	June, 1941	
Discussion	Sept., Oct., Dec., 1941	Jan., 1942
Cost of Public Services in Residential Areas. *F. Dodd McHugh*	June, 1941	
Discussion	Oct., Nov., 1941	Jan., 1942
Method of Predicting the Runoff from Rainfall. *Ray K. Linsley, Jr., and William C. Ackermann*	June, 1941	
Discussion	Oct., Nov., Dec., 1941	Jan., 1942
An Investigation of Plate Girder Web Splices. *J. M. Garrelts and I. E. Madsen*	June, 1941	
Discussion	Sept., Oct., 1941	Jan., 1942
A Direct Method of Flood Routing. *C. O. Wisler and E. F. Brater*	June, 1941	
Discussion	Oct., Nov., 1941	Jan., 1942
Land Surveys and Titles: Second Progress Report of the Joint Committee of the Real Property Division, American Bar Association, and the Surveying and Mapping Division, American Society of Civil Engineers	June, 1941	
Discussion	Oct., Nov., 1941	Jan., 1942
Design and Construction of San Gabriel Dam No. 1. *Paul Baumann*	Sept., 1941	
Discussion	Nov., Dec., 1941	Jan., 1942
Pipe-Line Flow of Solids in Suspension: A Symposium	Oct., 1941	
Discussion	Oct., Dec., 1941	Feb., 1942
Fundamental Aspects of the Depreciation Problem: A Symposium	Nov., 1941	
Discussion	Nov., Dec., 1941	Mar., 1942
Stability of Granular Materials. *R. E. Glover and F. E. Cornwell*	Nov., 1941	
Discussion	Nov., 1941	Mar., 1942
Timber Friction Pile Foundations. *Frank M. Masters*	Nov., 1941	Mar., 1942
Bending Moments in the Walls of Rectangular Tanks. *Dana Young*	Nov., 1941	Mar., 1942
Energy Loss at the Base of a Free Overfall. *Walter L. Moore*	Nov., 1941	Mar., 1942

NOTE.—The closing dates herein published are final except when names of prospective discussers are registered for special extension of time.

* Publication of closing discussion pending.

CONTENTS FOR DECEMBER, 1941

PAPERS

PAGE

Design of St. Georges Tied Arch Span.
By J. M. Garrelts, Assoc. M. Am. Soc. C. E. 1801

Allocation of the Tennessee Valley Authority Projects.
By Theodore B. Parker, M. Am. Soc. C. E. 1813

Development of Transportation in the United States.
By J. E. Teal, M. Am. Soc. C. E. 1827

Lateral Stability of Unsymmetrical I-Beams and Trusses in Bending.
By George Winter, Esq. 1851

DISCUSSIONS

Fundamental Aspects of the Depreciation Problem: A Symposium.
By Messrs. William G. Atwood, George E. Goldthwaite, Nathan B. Jacobs, H. J. Flagg, Nelson Lee Smith, and C. Beverley Benson 1865

Traffic Engineering as Applied to Rural Highways.
By Messrs. Nathan Cherniack, and Park H. Martin 1879

Design and Construction of San Gabriel Dam No. 1.
By William P. Creager, M. Am. Soc. C. E. 1886

Fort Peck Slide.
By Irving B. Crosby, Affiliate Am. Soc. C. E. 1887

Value of Public Works.
By J. P. Hallihan, M. Am. Soc. C. E. 1889

Concrete in Sea Water: A Revised Viewpoint Needed.
By Homer M. Hadley, Assoc. M. Am. Soc. C. E. 1893

Crack Prevention Program, Hiwassee Dam.
By O. Laurgaard, M. Am. Soc. C. E. 1910

CONTENTS FOR DECEMBER, 1941 (*Continued*)

PAGE

Formulas for the Transportation of Bed Load.
By H. A. Einstein, Assoc. M. Am. Soc. C. E. 1917

Design of Acceleration and Deceleration Lanes.
By Adolphus Mitchell, Assoc. M. Am. Soc. C. E. 1921

Surface Runoff Determination from Rainfall Without Using Coefficients.
By W. I. Hicks, Esq. 1925

Evaluation of Flood Losses and Benefits.
By Malcolm Elliott, M. Am. Soc. C. E. 1933

Method of Predicting the Runoff from Rainfall.
By Franklin F. Snyder, Jun. Am. Soc. C. E. 1935

Pipe-Line Flow of Solids in Suspension: A Symposium.
By H. A. Einstein, Assoc. M. Am. Soc. C. E. 1937

Pile-Driving Formulas: Progress Report of the Committee on the Bearing Value of Pile Foundations.
By Messrs. Trent R. Dames and William W. Moore, Maxwell M. Upson, Gregory P. Tschebotarioff, Robert F. Legget, and Jacob Feld 1939

Salts in Irrigation Water.
By Messrs. Louis J. Alexander, W. D. Collins, and John H. Bliss 1957

The Surveyor and the Law.
By D. D. Haines, Assoc. M. Am. Soc. C. E. 1968

INDEXES

Subject and Author Indexes, 1941 *Proceedings* 1971

For Index to all Papers, the discussion of which is current in PROCEEDINGS, *see page 2*

AMERICAN SOCIETY OF CIVIL ENGINEERS

Founded November 5, 1852

PAPERS

DESIGN OF ST. GEORGES TIED ARCH SPAN

By J. M. Garrelts,[1] Assoc. M. Am. Soc. C. E.

Synopsis

The procedure used to determine the preliminary and final designs for a tied arch bridge is presented in this paper. Although the type of structure is not new, having been used in Europe, it is believed that the design introduces a new type in American practice. The structure differs from the usual tied arch in that the ratio of the moment of inertia of the tie girder to that of the arch rib is approximately 13 to 1. With these relative physical properties, the total bending moment taken by the rib and girder together is divided between these two elements, the major part being resisted by the girder. (The distribution of this bending moment has been discussed elsewhere by Nathan M. Newmark,[2] Assoc. M. Am. Soc. C. E.)

Introduction

The St. Georges bridge (Fig. 1) is designed to carry the DuPont Highway over the Chesapeake and Delaware Canal at St. Georges, Del. It consists of a 540-ft span over the canal flanked on either side by steel plate-girder and rolled-beam approaches, the over-all length between abutments being approximately 4,200 ft. Selection of a suitable type of structure for the main channel span involved both the essential provision for clearance and a solution that would be satisfactory from the esthetic standpoint.

Provision for full clearance below the floor level for the entire span limited the possible types of structure. Studies made by the architect and the engineers resulted in the selection of a tied arch. With this type, it was possible to extend the main line of the approach girders through the tie girder of the arch and thus, through this esthetically dominating line in the design, to give a continuity, a feeling of unity, to the entire construction. Further emphasis of this main element of the esthetic composition was secured by designing the arch

Note.—Written comments are invited for immediate publication; to insure publication the last discussion should be submitted by **April, 1942.**

[1] Associate Prof., Civ. Eng., Columbia Univ., and Designing Engr., Waddell & Hardesty, New York, N. Y.

[2] *Transactions*, Am. Soc. C. E., Vol. 103 (1938), p. 73.

rib itself relatively thin, making it appear more as a reinforcement for the tie girder than as the main supporting member.

The writer realizes that this basic idea in the esthetics of the structure may be questioned—other approaches were possible—but, after due consideration, this plan of treatment seemed to be the most promising. It is not the purpose of this paper, however, to develop this phase of the St. Georges design in greater detail.

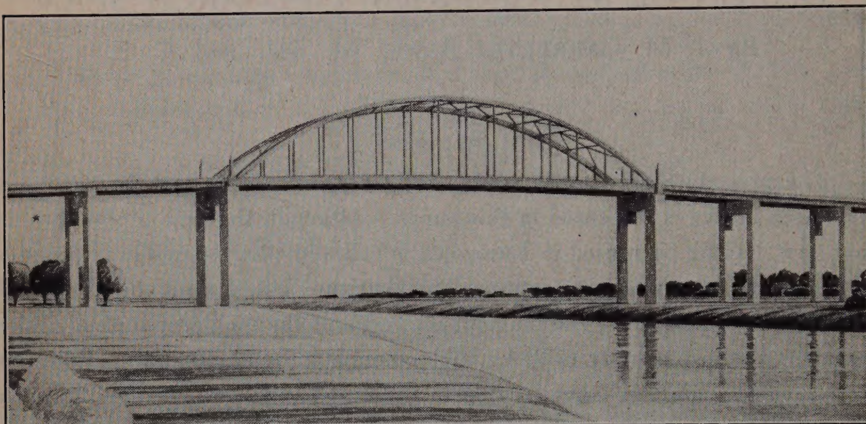


Fig. 1.—St. Georges Highway Bridge

In line with this adopted plan, the tie girder was made the same depth as the approach girders—about 9 ft; and a depth of 3.5 ft was selected for the arch rib. With these dimensions, the tie girder will resist the major part of the bending moments produced by the live load.

Such a structure is multiply indeterminate, and it is the purpose of this paper to outline the method used in arriving at the final design and to describe the completed structure.

Notation

The letter symbols in this paper are defined where they first appear and are assembled in the Appendix for convenience of reference.

Preliminary Design

By making some reasonable assumptions which did not introduce an appreciable error in the analysis, it was possible to simplify the problem considerably. In the particular structure being considered, the hangers were rather long and slender, and it was assumed that they would not resist bending moment but would carry only direct stress.

As a first approximation, in the preliminary analysis, it was further assumed that the arch rib resisted no bending moment—that it was subjected only to compression. This assumption implies that the tie girder resists the tension produced in the tie, and also a moment at any vertical section that is equal to the difference between the bending moment due to the external loads and the

moment equal to the horizontal component (H) of force in the rib and girder times the vertical distance between the neutral axes of the girder and rib. This can be written as follows (see Fig. 2(*a*)):

$$M = M' - H\,y \quad \text{(1)}$$

in which: H = horizontal component of the rib compression = horizontal component of the tie tension; M = bending moment to be resisted by the tie girder; M' = simple beam bending moment at any vertical cross section A–A (Fig. 2(*a*)) produced by the external forces. The horizontal component of the rib

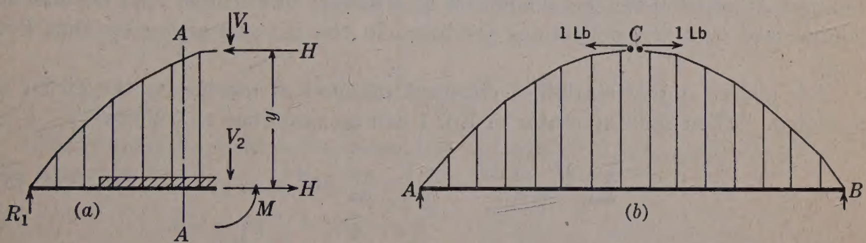


Fig. 2

compression is taken as the redundant. To determine this force H, a statically determinate substitute structure, obtained by cutting the rib at the center C (Fig. 2(*b*)), is assumed. The principal of virtual work can be used to determine H.

Let: Δ'_C = relative horizontal displacement of the two cut faces at C, due to the applied load, with $H = 0$; Δ_{1C} = relative horizontal displacement of the two cut faces at C, due to $H = 1$; then

$$H = -\frac{\Delta'_C}{\Delta_{1C}} \quad \text{(2)}$$

in which

$$\Delta'_C = \sum \frac{M''\,m\,\Delta l}{E\,I} + \sum \frac{s''\,u\,l}{A\,E} \quad \text{(3a)}$$

and

$$\Delta_{1C} = \sum \frac{m^2\,\Delta l}{E\,I} + \sum \frac{u^2\,l}{A\,E} \quad \text{(3b)}$$

M'' being the bending moment and s'' the direct stress in any member of the substitute structure ($H = 0$), produced by the applied loads. The symbol m is the bending moment and u the direct stress in any member of the structure due to $H = 1$; l and Δl are the length of any member and an element of length of a member, respectively, used in the summation; E is the modulus of elasticity; A, the cross-sectional area; and I, the moment of inertia of the cross-sectional area of any member.

The formula for H can be written:

$$H = -\frac{\sum \dfrac{M''_G\,m\,\Delta l}{E\,I_G}}{\sum \dfrac{m^2\,\Delta l}{E\,I_G} + \sum \dfrac{u^2\,\Delta l}{A_G\,E} + \sum \dfrac{u^2\,\Delta l}{A_R\,E}} \quad \text{(4)}$$

in which A_G and I_G are the area and the moment of inertia of the cross section of the girder, respectively, and A_R is the area of the rib.

To be consistent with the assumption that compression alone acts in the rib, the hanger loads are assumed to be equal, and the load points or points of intersection of the hangers and rib are selected so that they lie on a parabolic curve.

The effects of any deformations in the hangers are neglected. This approximation introduces very small error because the hangers can be fabricated so that they will have the correct length under dead load; and at any section where the moment is appreciable the elongation of a hanger due to live load is small in comparison with the deflections produced in the rib and girder by that live load.

For a first approximation, a constant moment of inertia for the girder is assumed. Then the numerator of Eq. 1 can be simplified as follows:

$$\sum \frac{M'' \, m \, \Delta l}{E \, I_G} = \frac{1}{E \, I_G} \sum M'' \, m \, \Delta l \qquad (5)$$

Due to $H = 1$, the load in the hangers is equivalent to $\frac{8 \, h}{L^2}$ per unit of length. Then

$$m = -\frac{4 \, h}{L} x + \frac{8 \, h \, x^2}{L^2 \; 2} = - y \qquad (6)$$

using the coordinates indicated in Fig. 3. For the girder $u = 1$, and for the rib $u = 1 \sec \alpha$.

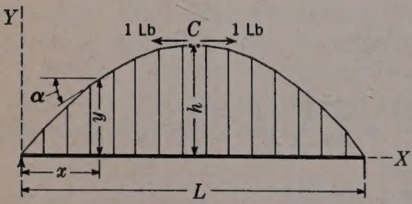


FIG. 3

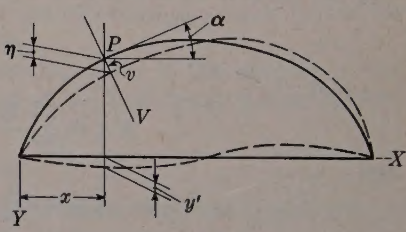


FIG. 4

The summations indicated in Eq. 4 reduce to the following values:

$$\sum \frac{m^2 \, \Delta l}{E \, I_G} = \frac{1}{E \, I_G} \sum y^2 \, \Delta l = \frac{8}{15} \frac{h^2 \, L}{E \, I_G} \qquad (7a)$$

$$\sum \frac{u^2 \, \Delta l}{E \, A_G} = \frac{1}{E \, A_G} \sum u^2 \, \Delta l = \frac{L}{E \, A_G} \qquad (7b)$$

and

$$\sum \frac{u^2 \, \Delta l}{E \, A_R} = \frac{1}{E \, A_R} \sum \left(1 \times \frac{\Delta l}{\Delta x}\right)^2 \Delta l = \frac{L + 8 \frac{h^2}{L}}{E \, A_R} \qquad (7c)$$

In Eq. 7c, A_R is assumed to be constant for this preliminary analysis.

From Eq. 4:

$$H = \frac{\int M'' \, y \, dl}{\frac{8}{15} h^2 L + \frac{I_G L}{A_G} + I_G \frac{\left(L + 8 \frac{h^2}{L}\right)}{A_R}} \qquad \text{(8a)}$$

The denominator of Eq. 8*a* depends only on the dimensions of the structure, and an examination of the terms indicates that the first term is large in comparison with the other two. Using only the first term, a simplified formula for H is obtained:

$$H = \frac{15}{8 \, h^2 L} \int M'' \, y \, dl \qquad \text{(8b)}$$

The bending moment to be resisted by the girder at any section is given by the equation

$$M_G = M'' - H \, y = M'' - \frac{15}{8} \frac{y}{h^2 L} \int M'' y \, dx \qquad \text{(9)}$$

Using Eq. 8*b*, influence values for H were determined by placing a unit load at each panel point in turn, from which the maximum values for the direct stresses in the rib and girder were computed. Influence values for M_G due to unit load at each panel point were computed, using Eq. 9, and with the aid of these coefficients the maximum live-load moments in the girder were determined. With these preliminary direct stresses and moments, sections were determined which were used as a basis for making the more exact analysis.

The total bending moment M, which was assumed to be resisted entirely by the tie girder in the preliminary analysis, is actually taken by both the rib and girder. It is necessary then to develop a method for determining the proportion of this total moment taken by the rib and by the girder.

In order to proportion this bending moment at any section between the arch rib and tie girder, the relation between the vertical components of deflection of the rib and girder at any section will be used.

At any point P in the arch rib (Fig. 4), let v represent the component of deflection along a normal to the arch rib curve at that point. From the differential equation for the deflection of a curved beam, the normal component of deflection v is

$$\frac{d^2 v}{dl^2} + \frac{v}{\rho^2} = \pm \frac{M_R}{E \, I_R} \qquad \text{(10a)}$$

in which dl is an element of length of the arch rib; ρ is the radius of curvature of the rib at point P; M_R is the moment taken by the rib; and I_R is the moment of inertia of the rib.

The second term of Eq. 10*a* can be neglected without appreciable error, and the equation reduces to

$$\frac{d^2 v}{dl^2} = \pm \frac{M_R}{E \, I_R} \qquad \text{(10b)}$$

The vertical component of the deflection of the arch rib is equal to the vertical component of v, if the tangential component of deflection at P (which is small) is neglected; then

$$\eta = v \cos \alpha \qquad (11)$$

in which η is the vertical component of v, and α is the angle between the tangent at P and the x-axis.

The first term of Eq. 10*b* can be replaced by an expression involving η, x, and α as follows: From Eq. 11—

$$\frac{d\eta}{dl} = \frac{dv}{dl} \cos \alpha - v \frac{d\alpha}{dl} \sin \alpha; \quad \text{and} \quad \frac{d^2\eta}{dl^2} = \cos \alpha \frac{d^2v}{dl^2}$$

$$- 2 \frac{dv}{dl} \frac{d\alpha}{dl} \sin \alpha - v \frac{d^2\alpha}{dl^2} \sin \alpha - v \left(\frac{d\alpha}{dl}\right)^2 \cos \alpha$$

$$= \cos \alpha \frac{d^2v}{dl^2} - 2 \frac{dv}{dl} \frac{1}{\rho} \sin \alpha + \frac{v}{\rho^2} \frac{d\rho}{dl} \sin \alpha - \frac{v}{\rho^2} \cos \alpha \qquad (12)$$

Neglecting the last three terms of Eq. 12:

$$\frac{d^2v}{dl^2} = \sec \alpha \frac{d^2\eta}{dl^2} \text{ (approximately)} \qquad (13)$$

Using $dx = ds \cos \alpha$, Eq. 13 reduces to

$$\frac{d^2v}{dl^2} = \cos \alpha \frac{d^2\eta}{dx^2} \qquad (14a)$$

or

$$\pm \frac{M_R}{E\, I_R} = \cos \alpha \left(\frac{d^2\eta}{dx^2}\right) \text{ (approximately)} \qquad (14b)$$

For the girder (Fig. 4)

$$\pm \frac{M_G}{E\, I_G} = \frac{d^2y'}{dx^2} \qquad (15)$$

in which M_G is the moment taken by the girder, and I_G is the moment of inertia of the girder.

The total moment M_T is equal to $M_R + M_G$; and, neglecting the change in lengths of the hangers, $\frac{d^2\eta}{dx^2} = \frac{d^2y'}{dx^2}$. Therefore, using Eqs. 14*b* and 15:

$$M_T = M_G \frac{I_G + I_R \cos \alpha}{I_G}$$

Then

$$M_G = M_T \left(\frac{I_G}{I_G + I_R \cos \alpha}\right) \qquad (16a)$$

and

$$M_R = M_T \left(\frac{I_R \cos \alpha}{I_G + I_R \cos \alpha} \right) \quad \text{(16b)}$$

The formula for H becomes

$$H = \frac{\sum \frac{M' \, m \, \Delta l}{I_{tr}}}{\sum \frac{m^2 \, \Delta l}{I_{tr}} + \sum \frac{\Delta l}{A_G} + \sum \frac{\sec^2 \alpha \, \Delta l}{A_R}} \quad \text{(17)}$$

in which $I_{tr} = I_G + I_R \cos \alpha$.

The Final Design

The span of the arch is 540 ft, with fifteen panels at 36 ft, and the rise is 100 ft. To perform the summations indicated in Eq. 17 the structure was divided into sections, each having a length equal to a panel length. With this equation, influence lines for H were determined as follows:

Load at panel point	Value of H
1	0.2358
2	0.4595
3	0.6625
4	0.8372
5	0.9772
6	1.0772
7	1.1300

The tie girder, instead of being straight, follows the vertical curve of the roadway, and the factor m in Eq. 17 is given by $m = y' - y$, in which y is the ordinate to the arch rib and y' is the ordinate to the girder.

The moment M_T to be resisted by the girder and arch rib together is given by the expression (see Eq. 1)

$$M_T = M' - H (y - y') \quad \text{(18)}$$

in which M' is the simple beam bending moment. The curves of Fig. 5 show the influence lines for M_T as computed for the final design sections. The maximum live-load moment was computed for the final design sections, and the proportionate part taken by both the rib and the girder determined in accordance with Eqs. 16. Shears were calculated but were found to be very small.

The dead load used for design was 15.1 kips per ft of length of bridge and the H-20 live load was used. The dimensions of the structure are shown in Fig. 6 and an example of the arrangement of the design stresses and sections is shown in Figs. 7 and 8. A dead-load moment of 500 ft-kips was included in the stresses at each point to allow for the fact that there may be some dead-load moment in the structure after it is under full dead load.

The total weight of steel in the span is 2,166 tons, classified as follows:

Members	Tons
Girders	711
Ribs	547
Hangers	123
Bracing	186
Floor beams	210
Stringers	291
Diaphragms and railing	98

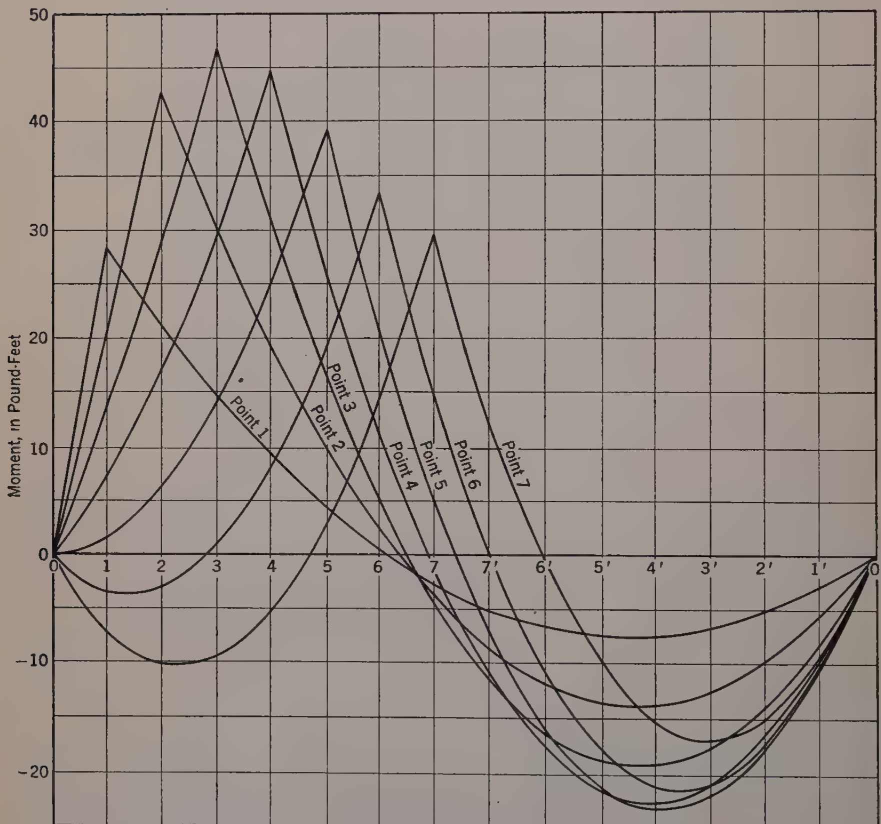


FIG. 5.—INFLUENCE LINES FOR MOMENT M_T AT PANEL POINTS

In the preceding analysis, the bending resistance of the rib has been transferred to the girder, giving a transformed structure in which the rib takes only compression whereas the tie takes tension and resists the total bending moment with a transformed moment of inertia. Theoretically, it would also be possible to analyze the structure as one in which the bending resistance of the girder is transferred to the rib. This would result in a transformed structure in which the tie girder would take only tension, while the rib would take compression

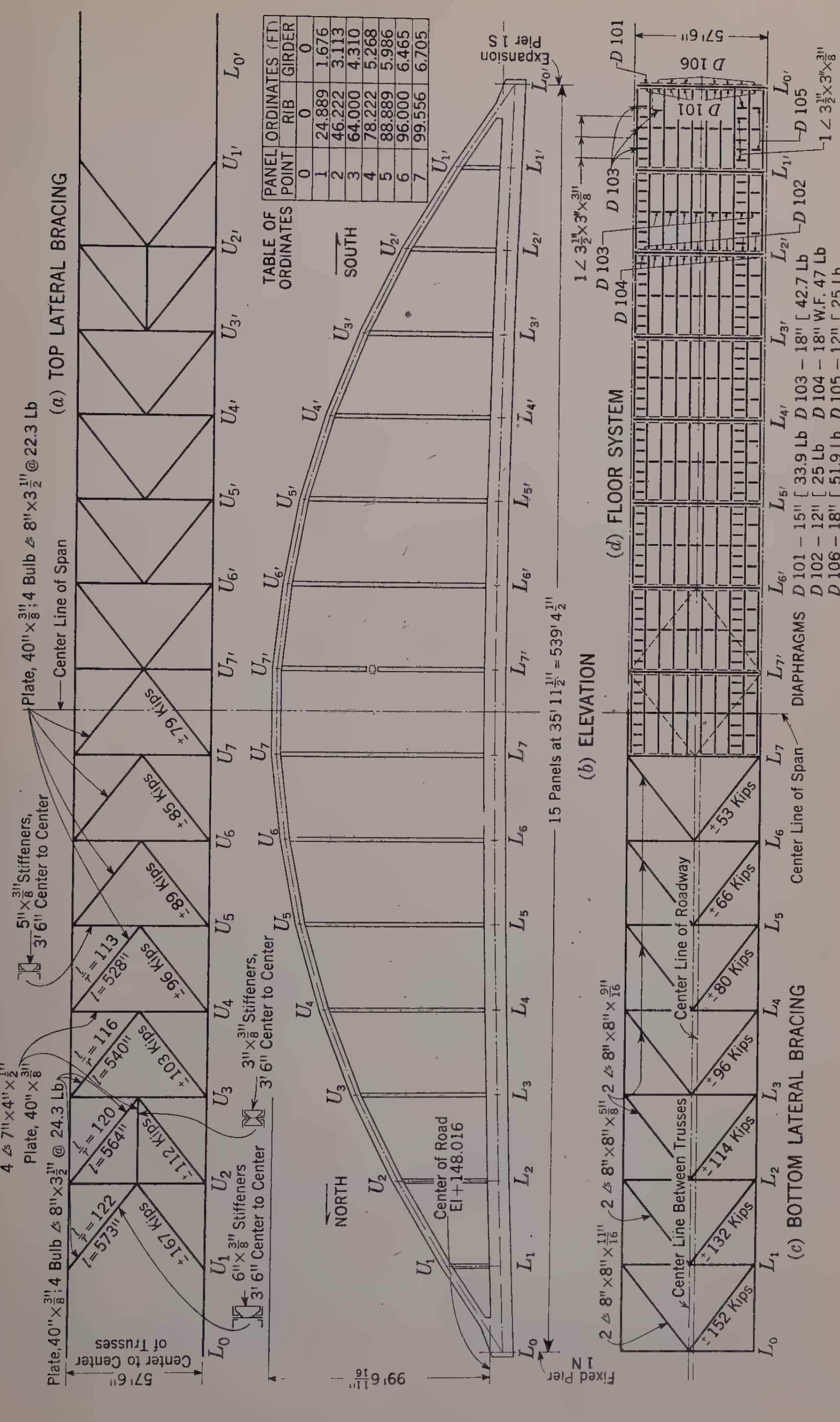


FIG. 6.—DIMENSIONS OF THE STRUCTURE

and would resist the total bending moment with a transformed moment of inertia $I'_{tr} = I_R + I_G \sec \alpha$.

SECTION	0-1		1-2		2-3		
POINT	1		1		3		
SECTION 	1 Cov. 39×15/16	36.56□"	1 Cov. 39×15/16	36.56□"	1 Cov. 39×15/16	36.56□"	1 Cov.
	2L 5×5×5/8	11.72	2L 5×5×5/8	11.72	2L 5×5×9/16	10.62	2L 5
	2 Webs 40×13/16	65.00	2 Webs 40×13/16	65.00	4 Webs 40×13/16	130.00	2 We
	2 Webs 40×3/4	60.00	2 Webs 40×3/4	60.00			2 W
	2 Pls. 27×13/16	43.87	2 Pls. 27×9/16	30.38			
	2L 8×6×13/16	21.44	2L 8×6×13/16	21.44	2L 8×6×3/4	19.88	2L
	1 Cov. 41×13/16 (27" Eff.)	21.95	1 Cov. 41×13/16 (27" Eff.)	21.95	1 Cov. 41×13/16 (27" Eff.)	21.95	1 Co
Gross Area, (□")	260.54		247.05		219.01		
Section Modulus	Top-2705	Bott. 2530	Top-2620	Bott. = 2500	Top-2550	Bott. - 2410	Top-
l/r	527 ÷ 13.92 = 37.7		503 ÷ 13.82 = 36.3		482 ÷ 13.67 = 35.3		
f Allowable (K/□")	19.34		19.40		19.44		
	Thrust	Moment	Thrust	Moment	Thrust	Moment	T
D. (Kips)	3560	± 43	3410	± 35	3270	± 33	
L.+I.; (+Moment)	210	+ 392	202	+ 328	276	+ 566	
L.+I.; (−Moment)	558	− 308	530	− 255	458	− 468	
Total D+L.+I.; (+Mom.)	3770	+ 435	3612	+ 363	3546	+ 599	
D.+L.+I.; (−Mom.)	4118	− 351	3940	− 290	3728	− 501	
Wind	173	90x-x; 1088y-y	82	74x-x; 1088y-y			
D.+L.+I.+W. (+Moment)	3943		3694				
D.+L+I+W. (−Moment)	4291		4022				
Unit Stresses	+ Moment	− Moment	+ Moment	− Moment	+ Moment	− Moment	+
D.+L.+I. Direct	14490	15820	14610	15950	16200	17020	
D.+L.+I. Bending	1930	1660	1660	1400	2820	2490	
Total	16420	17480	16270	17360	19020	19510	
Wind Direct	660	660	330	330			
Bending	5780	5810	6010	6030			
D.+L.+I.+W., Direct	15150	16480	14940	16280			
D.+L.+I.+W., Bending	7710	7470	7670	7430			
Total *	22,860	23,950	22,610	23,710			

* 25% increase in allowable unit stress.

SECTION	0-1		1-2		2-3		
POINT	1		2		3		
SECTION 	1 Cov. 25×3/4	18.75 gr. 15.37 nt.	1 Cov. 25×3/4	18.75 gr. 15.37 net	2 Cov. 25×5/8	31.25 25.62	2 Cov
	2 Cov. 25×13/16	40.63 33.30	2 Cov. 25×13/16	40.63 33.30	2 Cov. 25×11/16	34.39 28.20	2 Co
	2L 8×8×1	30.00 25.50	2L 8×8×1	30.00 25.50	2L 8×8×1	30.00 25.50	2L
	2 Webs 108×1/2	108.00 81.00	2 Webs 108×1/2	108.00 81.00	2 Webs 108×1/2	108.00 81.00	2 W
	2L 8×6×3/4	19.88 16.51	2L 8×6×3/4	19.88 16.51	2L 8×6×3/4	19.88 16.51	2L
	1 Cov. 39×5/8	24.38 20.16	1 Cov. 39×5/8	24.38 20.16	1 Cov. 39×11/16	26.81 22.17	1 Co
	2 Cov. 39×9/16	43.87 36.28	2 Cov. 39×9/16	43.87 36.28	2 Cov. 39×5/8	48.75 40.31	2 Co
Area (□")	285.51 gr.	228.13 net	285.51 gr.	228.13 net	299.08 gr.	239.31 net	29
Section Modulus	Top-9100 net	Bott. = 9130 net	Top-9100 net	Bott.-9130 net	Top-9640 net	Bott-9770 net	Top-9
	Tension	Moment	Tension	Moment	Tension	Moment	Te
D. (Kips)	2930	± 458	2930	± 465	2930	± .467	
L.+I.; (+Moment)	173	+ 4350	233	+ 6850	247	+ 8100	
L.+I.; (−Moment)	456	− 3390	411	− 5720	411	− 6700	
Total D.+L.+I; (+Mom)	3102	+ 4808	3163	+ 7315	3177	+ 8567	
D.+L+I.,(−Mom.)	3386	− 3848	3341	− 6185	3341	− 7167	
Unit Stresses	+ Moment	− Moment	+ Moment	− Moment	+ Moment	− Moment	+
D.+L.+I. Direct	13600	14850	13860	14650	13280	13970	
D.+L.+I. Bending	6320	5080	9620	8170	10530	8920	
Total	19920	19930	23480	22820	23810	22890	

FIG. 7.—EXAMPLE OF THE ARRANGEMENT OF DATA: DESIGN STRESSES AND SECTIONS

The proper procedure in any case should be that which more nearly agrees with the behavior of the structure. In the St. Georges bridge, the girder resists approximately 93% of the total bending moment and the rib 7%, so

that the procedure used is more nearly in agreement with the structural action which the analysis indicates.

The tie girder and arch rib are both cambered for full dead load. However, the dead-load moment in the tie girder will not be zero unless the span is riveted up in the position it will assume under full dead load. Due to the fact

HANGERS

MEMBER	U_1L_1	U_2L_2	U_3L_3 U_4L_4 U_5L_5	U_6L_6 U_7L_7
D. (Kips)	230	230	230	230
L.	79	79	79	79
I.	6	6	6	6
D. + L. + I.	315	315	315	315
l/r	204 ÷ 5.10 = 40	516 ÷ 5.14 = 101	994 ÷ 5.93 = 168	1112 ÷ 6.69 = 166
f Allowable	24000#/□"	18000#/□"	18000#/□"	18000#/□"
Section Area Req.	13.12 □" net	17.50 □" net	17.50 □" net	17.50 □" net
Section Used	2-12" Ship [30.9	2-12" Ship [30.9	2-13" Car [31.8	2-15" [33.9
	2 Pls. 23×½ (17" Eff.)	2 Pls. 23×½ (17" Eff.)	2 Pls. 23×½ (17" Eff.)	2 Pls. 23×½ (17" Eff.)
Area □"	35.00 gr. 26.80 nt.	35.00 gr. 26.80 nt.	35.60 gr. 27.85 net	36.80 gr. 28.10 net
Material	Silicon	Carbon	Carbon	Carbon

NOTE: Hangers participate partially with rotation of floorbeams due to Live Load.

FIG. 8.—EXAMPLE OF ARRANGEMENT OF DATA: CHARACTERISTICS OF HANGERS

that the riveting up will be done before the concrete deck is poured, some method of closing is necessary which will induce a moment in the girder at the time of riveting to neutralize the moment produced by the additional dead load of the floor slab. This will be accomplished by closing the span on a pin in the tie girder, at the center of the span located above the center line of the girder a distance sufficient to develop the predetermined moment. By riveting this joint up in the distorted position, the desired result will be obtained.

A model analysis was made of the St. Georges Arch by graduate students at Princeton University at Princeton, N. J., under the direction of Prof. E. K. Timby, Assoc. M. Am. Soc. C. E. The "Beggs Deformeter" was used to measure the strains in a celluloid model. Influence values for both the direct stress and the moment computed from the deformeter measurements were in close agreement with those calculated by the theoretical analysis. The results of these tests serve to confirm the correctness of the assumptions made in the design analysis.

ACKNOWLEDGMENTS

The St. Georges Bridge is a project of the Corps of Engineers of the U. S. Army under the supervision of Lt.-Col. Harry B. Vaughan, Jr., M. Am. Soc. C. E., district engineer of the Philadelphia District.

Maurice N. Quade, M. Am. Soc. C. E., is in charge of the project for the firm of Parsons, Klapp, Brinckerhoff and Douglas; Alfred Hedefine, Assoc. M. Am. Soc. C. E., is in charge of the design of the arch span for the firm of Waddell and Hardesty; and Aymar Embury II, M. Am. Soc. C. E., is the consulting architect.

APPENDIX

Notation

The following symbols, used in this paper, conform essentially with American Standard Symbols for Structural Analysis, prepared by a Committee of the American Standards Association, with Society cooperation, and approved by the Association in 1941:[3]

A = area of cross section:
- A_G = area of girders;
- A_R = area of rib;

E = modulus of elasticity;

H = horizontal component of rib compression and tie tension;

h = rise of the arch rib;

I = moment of inertia:
- I_G = of girder;
- I_R = of rib;

L = length of span;

l = length of panel, or other structural member; Δl = a segment of l;

M = bending moment to be resisted by the tie girder:
- M' = simple beam moment at any vertical cross section (section A–A, Fig. 2(a)) produced by the external forces;
- M'' = moment in any member of a substitute structure;
- M_G = moment to be resisted by the girder at any section;
- M_R = moment resisted by the rib;
- M_T = total moment;

m = bending moment in any member of a structure due to $H = 1$;

s'' = stress in the substitute structure ($H = 0$), produced by the applied loads;

u = direct stress in any member of a structure, due to $H = 1$;

v = normal component of the deflection;

x = horizontal coordinate distance to any point on arch rib or girder;

y = vertical ordinate to any point on arch rib;

y' = vertical ordinate to any point on tie girder;

Δ = total deformation = relative horizontal displacement of two cut faces:
- Δ' = displacement when $H = 0$;
- Δ_1 = due to $H = 1$;

η = vertical component of v;

α = slope of the arch rib at any point = angle between the tangent at point P, and the x-axis;

ρ = the radius of curvature of the rib at point P.

---

[3] Publication pending.

AMERICAN SOCIETY OF CIVIL ENGINEERS

Founded November 5, 1852

PAPERS

ALLOCATION OF THE TENNESSEE VALLEY AUTHORITY PROJECTS

By Theodore B. Parker,[1] M. Am. Soc. C. E.

Synopsis

The various theories applicable to the allocation of costs of a multiple-purpose water control project, and the procedure followed by the Tennessee Valley Authority in allocating the cost of its first three dams to navigation, flood control, and power, are presented in this paper. It is shown that a large difference in the allocation of the common costs of a project affects to a much lesser degree the proportions of the total cost charged to the individual purposes. Consequently, the final allocations are approximately correct, within reasonable limits, regardless of the exact method adopted.

Requirements of the Act

The allocation problem of the Tennessee Valley Authority (TVA) was necessitated by Sections 9*a* and 14 (see Reference 1 in the Appendix) of the Tennessee Valley Authority Act as amended. It is to be noted at the outset that Section 9*a* specifically provides for the generation of power only in so far as it is consistent with the primary purposes of navigation and flood control and that the proceeds of the sale of power are "to assist in liquidating the cost or aid in the maintenance of the projects of the Authority."

The location and the minimum pool elevations of the Authority's dams on the main Tennessee River (Fig. 1) were determined by navigation requirements, whereas the surcharge pool elevations were determined by the maximum amount of flood control storage space consistent with a minimum damage to the cities and agricultural areas. Dams on the tributary rivers were chosen principally with regard to flood control purposes. All of these dams operating together as a unified power system, in so far as possible after satisfying the primary purposes of navigation and flood control, produce a large amount of power. Nevertheless, the intent of the Act is clear and is rigidly followed in actual practice and in the operation of the Authority's system.

Note.—Written comments are invited for immediate publication; to insure publication the last discussion should be submitted by **April, 1942**.

[1] Chf. Engr., TVA, Knoxville, Tenn.

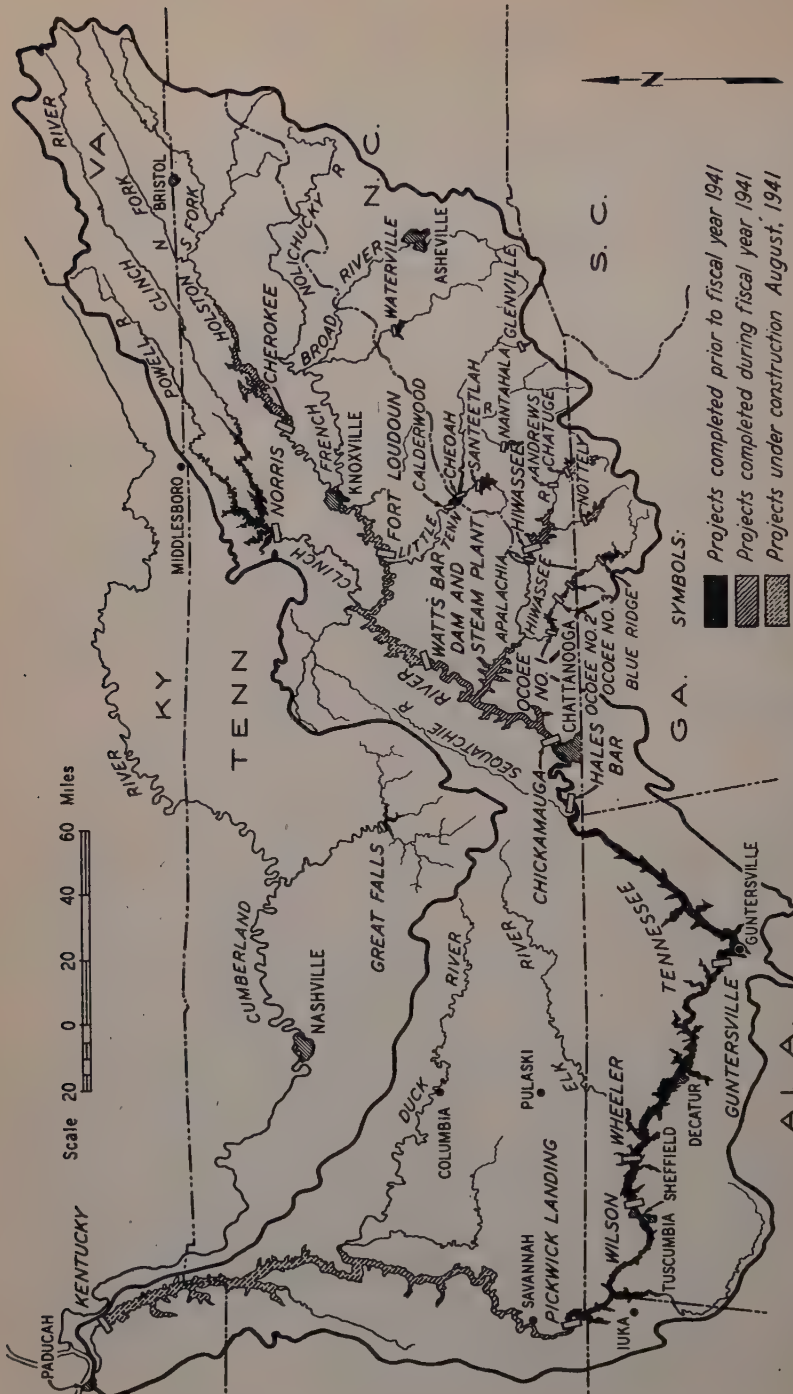


Fig. 1.—Tennessee River Valley

With regard to the allocation problem, two reasonable points of view could be taken. The power generated by the different dams could be considered only as by-product power. In Section 14 (see Reference 1 in the Appendix) the Act declares

> "* * * as soon as practicable to make the power projects self-supporting and self-liquidating the surplus power shall be sold at rates which in the opinion of the board, when applied to the normal capacity of the Authority's power facilities, will produce gross revenues in excess of the cost of production of said power."

This might be taken to mean that the rates should be such that they will pay only for all additional facilities especially necessary to install power equipment at the Authority's dams together with all transmission facilities without, however, paying any share of the dams themselves. A second interpretation would include in the cost of production a share of the cost of the facilities used jointly with navigation and flood control.

In July, 1940, Congress appropriated funds for the construction of the Cherokee hydroelectric project, the Watts Bar Steam Plant, and the installation of additional units at existing downstream plants. In July, 1941, four additional hydroelectric projects on the Hiwassee tributary and additional units in downstream plants were authorized. At present (October 20, 1941) Congress is considering the authorization of two additional tributary projects. These projects will be operated primarily for power production during the present national emergency, after which all of the projects will be operated on a multiple-purpose basis, and their total cost will be allocated to navigation, flood control, and power.

History of Procedure

In order to assist in the task of determining the proper theory and to make the necessary studies, the TVA Board of Directors created a Committee on Financial Policy consisting of the comptroller as chairman, the chief engineer, the chief water control planning engineer, the solicitor, the chief power planning engineer, and the chief budget officer. It was the opinion of this committee, acting with the advice of a number of consultants, that in order properly to assist "in liquidating the cost or aid in the maintenance of the projects of the Authority," a part of the joint costs should be allocated to power. In other words, although the committee recognized the limitation of the power program by priority given to navigation and flood control requirements in the planning and operation of the projects, the economies of the multiple-purpose developments are shared by all purposes.

In allocating a part of the joint costs to power the program was thereby placed on practically the same basis as (for instance) a private company that builds its own power facilities at an existing government navigation dam and pays the government a fee for the use of the head created by the dam. This fee may be considered to repay the government for a part of the cost of constructing the dam (see References 2 and 3). Still another comparison might

be made with a private company building a dam and power development on a navigable stream, the government building a navigation lock at the same location and granting the company a reduction or waiver of the license fee which would normally be required.

Such agreements between the U. S. Government and private companies are in effect examples of cost allocations, similar in character to the cost allocations at TVA dams.

At all times, the TVA Board of Directors was kept informed of the committee's work, and after the first "Report of the Investment in the Wheeler, Norris and Wilson Projects and the Allocation of the Investment Among the Several Purposes of the Tennessee Valley Authority Act," the committee submitted to the Board each year a report on the allocation of the Authority's investment in the system completed during the previous fiscal year. The first report, which was published as House Document No. 709, 75th Congress, 3d Session, included a full description of the committee's considerations leading to the allocation of the Authority's investment in the multiple-purpose projects to the various purposes.

Theories of Joint-Cost Allocations

The various theories of joint-cost allocations described in House Document No. 709 are summarized herein. It should be observed that allocations of costs are concerned principally with the proper apportionment of the cost of the structures and facilities which are used jointly for the various purposes of the development. Expenditures made solely for any single purpose are to be charged directly to that purpose. The following methods of allocation were considered by the committee.

Vendibility Theory.—Under competitive open market conditions, products jointly produced will generally be sold at prices totaling the joint cost of production plus a reasonable profit. Apportionment of costs will depend largely upon the demand for the products separately. This theory breaks down because there is no open market in which the services produced by the Authority under conditions of joint supply can be sold. Navigation and flood control, under present conditions, are not vendible commodities.

Benefit Theory.—This theory advocates apportionment of joint costs on the basis of estimated benefits derived from the completed development.

The difficulties peculiar to this theory are principally those of evaluation of the benefits. Beneficial effects of any one project are multitudinous. Some of these are gains to individual property owners, reduction in costs to consumers, profits to merchants and industrialists, advantages of a social and governmental character; some are specific, others general in nature; some are easily evaluated, others are intangible; some are local, others remote from the Valley; some are immediately effective, others will benefit future generations. House Document No. 709 presents this theory in some detail and furnishes much pertinent information.

It is necessary to distinguish between two versions of the benefit theory. According to one version, the benefits would include the over-all advantage

to the entire country from the projects constructed by the Authority. According to the other version, the benefits would be measured by the alternative cost of obtaining equivalent results by single-use projects, provided the advantages to the country can be shown to be sufficient to justify the expenditure necessary to construct such a single-use project. This second interpretation is probably the most workable of any of the methods that have been considered and is discussed in House Document No. 709 under the name of "alternative-justifiable-expenditure theory." A similar method was suggested for allocating joint costs between navigation and power for the St. Lawrence River project on the international section of that stream, and has also been used as a basis for allocating joint costs of water utilities between commercial service and fire-protection service.

Use of Facilities Theory.—This theory would distribute joint costs upon the basis of comparative use of the joint facilities. To each single function would be allocated such share of the joint cost as is measured by the extent of its use.

In the steam railroad field, for example, the apportionment of common cost between freight, mail, express, and passenger business may be made upon some comparable use units like car-mile, passenger-mile, or ton-mile measure.

When applied to the TVA problem, the theory becomes very involved; an acre-foot of reservoir capacity or an acre-foot of water released has been suggested as the common unit; but when considering power, head is an essential feature, whereas for flood control, head is not an essential factor. It is possible, however, that the theory might be usable after sufficient records have been collected over a considerable period of time regarding the actual use which has been made of the facilities. For the immediate future, the theory appears not to be workable.

Equal Apportionment.—This may be considered as a common-sense rule of equity to be used when it is felt that no truly scientific basis of apportionment can be found. However, such a rule does not seem practicable where the respective uses for each function are not equal.

Special Costs.—This theory suggests the apportionment of the costs to the several uses in direct proportion to the special expenditures made for each use. The procedure appears irrelevant where the object of the allocation is to impute responsibility for joint costs.

Alternative Justifiable Expenditure.—This theory proposes to divide the joint costs in proportion to the alternative justifiable expenditure less the direct cost for each function. By constructing projects which serve multiple uses, savings may be achieved in expenditures over those necessary for single-use projects.

According to the theory (see Reference 4 in the Appendix), the direct cost for any one purpose corresponds to the investment that could have been eliminated from the total project cost if that purpose had not been included in the development of the project. The alternative justifiable expenditure for any one purpose is the lowest cost of realizing an equal benefit to that obtained in the multiple-use project by a development undertaken solely for that purpose provided such expenditure is justified by the benefits obtainable.

Table 1 illustrates the application of this theory to the following simple hypothetical case—the assumed allocation of $1,000,000:

Assumed Direct Costs:		
Navigation	$ 150,000	
Flood control	100,000	
Power	250,000	
Total		$ 500,000
Common costs		500,000
Total project cost		$1,000,000

Assumed Alternative Costs:	
Navigation	$ 400,000
Flood control	300,000
Power	600,000
Total	$1,300,000

A conclusion as to the various allocation theories considered is given in the Appendix (see Reference 5).

TABLE 1.—Assumed Allocation of $1,000,000

Facility	Alter-native costs	Direct costs	Remaining costs; Col. 2 minus Col. 3	Percentage of total	Allocation of common costs	Direct costs added; Col. 3 plus Col. 6	Percentage of total
(1)	(2)	(3)	(4)	(5)	(6)	(7)	(8)
Navigation.......	$400,000	$150,000	$250,000	31.3	$156,000	$306,000	30.6
Flood control.....	300,000	100,000	200,000	25.0	125,000	225,000	22.5
Power...........	600,000	250,000	350,000	43.7	219,000	469,000	46.9
Total.........	$1,300,000	$500,000	$800,000	100.0	$500,000	$1,000,000	100.0

In Table 2, Items 1 to 21 comprise a summary of the trial computations based on the alternative-justifiable-expenditure method for the successive steps in the TVA program of construction up to the close of fiscal year 1940 and including the Wilson, Wheeler, Norris, Pickwick Landing, Chickamauga, Guntersville, and Hiwassee developments. In Table 2, Items 22 to 34 are a summary of the final allocations for this same program as recommended by the Financial Policy Committee. It will be seen that the latter vary from the trial computations in Table 2, Items 1 to 21, by relatively small sums. This variation represents the factor of judgment that entered into the committee's consideration of the problem in making their final determination, as is explained subsequently. (Costs used in the three-plant allocation exclude $4,774,905 reserved for Wilson Dam depreciation from date of completion to date of acquisition by the Authority. In all subsequent allocations, total costs have been used—reconstruction cost new at Wilson—and depreciation reserves at all projects have been set up in a special account, independent of allocations.)

TABLE 2.—System Allocations

No.	Facility	Three-Plant System, Including Norris, Wheeler, and Wilson Dams		Four-Plant System, Including Pickwick Landing Dam		Five-Plant System, Including Guntersville Dam		Six-Plant System, Including Chickamauga Dam		Seven-Plant System, Including Hiwassee Dam	
		Dollars	%	Dollars	%	Dollars	%	Dollars	%	Dollars	%
1	Total investment allocated	98,900,576	....	127,300,982	....	158,584,469	....	193,297,469	....	210,279,469	....
	Alternative Costs:										
2	Navigation	48,334,000	....	64,006,000	....	85,510,000	....	105,160,000	....	106,160,000	....
3	Flood control	33,210,000	....	38,447,000	....	44,435,000	....	52,310,000	....	63,260,000	....
4	Power	79,230,000	....	100,689,000	....	115,236,000	....	138,894,000	....	150,055,000	....
5	Total	160,774,000	....	203,142,000	....	245,181,000	....	296,364,000	....	319,475,000	....
	Direct Costs:										
6	Navigation	4,408,807	....	9,709,037	....	12,500,024	....	17,177,896	....	17,177,896	....
7	Flood control	2,600,000	....	3,621,000	....	3,621,000	....	4,661,000	....	4,871,000	....
8	Power	26,059,335	....	36,020,676	....	44,218,468	....	53,114,788	....	56,368,996	....
9	Total	33,068,142	....	49,350,713	....	60,339,492	....	74,953,684	....	78,417,892	....
	Remaining Alternative Costs:										
10	Navigation	43,925,193	34.4	54,296,963	35.3	73,009,976	39.5	87,932,104	39.7	88,982,104	36.9
11	Flood control	30,610,000	24.0	34,826,000	22.6	40,814,000	22.1	47,649,000	21.5	58,389,000	24.2
12	Power	53,170,665	41.6	64,668,324	42.1	71,017,532	38.4	85,779,212	38.8	93,686,004	38.9
13	Total	127,705,858	100.0	153,791,287	100.0	184,841,508	100.0	221,410,316	100.0	241,057,108	100.0
	Allocation of Common Costs:										
14	Navigation	22,643,459	34.4	27,516,445	35.3	38,806,766	39.5	46,982,483	39.7	48,656,922	36.9
15	Flood control	15,779,470	24.0	17,616,761	22.6	21,712,140	22.1	25,443,914	21.5	31,910,502	24.2
16	Power	27,409,505	41.6	32,817,063	42.1	37,726,071	38.4	45,917,388	38.8	51,294,153	38.9
17	Total	65,832,434	100.0	77,950,269	100.0	98,244,977	100.0	118,343,785	100.0	131,861,577	100.0
	Allocation of Total Costs:										
18	Navigation	27,052,266	27.3	37,225,482	29.2	51,306,790	32.3	64,160,379	33.2	65,834,818	31.3
19	Flood control	18,379,470	18.6	21,237,761	16.7	25,333,140	16.0	30,104,914	15.6	36,781,502	17.5
20	Power	53,468,840	54.1	68,837,739	54.1	81,944,539	51.7	99,032,176	51.2	107,663,149	51.2
21	Total	98,900,576	100.0	127,300,982	100.0	158,584,469	100.0	193,297,469	100.0	210,279,469	100.0
22	Total investment allocated	94,125,671	....	127,300,982	....	158,584,469	....	193,297,469	....	210,279,469	....
	Direct Costs:										
23	Navigation	4,075,988	....	9,709,037	....	12,500,024	....	17,177,896	....	17,177,896	....
24	Flood control	2,600,000	....	3,621,000	....	3,621,000	....	4,661,000	....	4,871,000	....
25	Power	23,967,177	....	36,020,676	....	44,218,468	....	53,114,788	....	56,368,996	....
26	Total	30,643,165	....	49,350,713	....	60,339,492	....	74,953,684	....	78,417,892	....
	Allocation of Common Costs:										
27	Navigation	22,218,877	35.0	28,062,097	36.0	38,315,541	39.0	46,154,076	39.0	47,470,168	36.0
28	Flood control	15,870,627	25.0	18,708,064	24.0	20,631,445	21.0	24,852,195	21.0	31,646,778	24.0
29	Power	25,393,002	40.0	31,180,108	40.0	39,297,991	40.0	47,337,514	40.0	52,744,631	40.0
30	Total	63,482,506	100.0	77,950,269	100.0	98,244,977	100.0	118,343,785	100.0	131,861,577	100.0
	Allocation of Total Costs:										
31	Navigation	26,294,865	27.9	37,771,134	29.7	50,815,565	32.0	63,331,972	32.7	64,648,064	30.7
32	Flood control	18,470,627	19.6	22,329,064	17.5	24,252,445	15.3	29,513,195	15.3	36,517,778	17.4
33	Power	49,360,179	52.5	67,200,784	52.8	83,516,459	52.7	100,452,302	52.0	109,113,627	51.9
34	Total	94,125,671	100.0	127,300,982	100.0	158,584,469	100.0	193,297,469	100.0	210,279,469	100.0

Comment

The allocation of the costs of the multiple-purpose projects is required by the TVA Act, which states that the cost of such projects shall be charged to flood control, navigation, fertilizer, national defense, and the development of power. It also requires that the findings of the allocation shall be used thereafter for the purpose of keeping the book value of these properties. Although not explicitly stated, it may be assumed that the reason Congress required such allocation was to determine equitable rates for the sale of power. This assumption is further borne out by the language of the same section of the Act concerning the keeping of accounts, which states that

> "for the purpose of accumulating data useful to the Congress in the formulation of legislative policy in matters relating to the generation, transmission, and distribution of electric energy * * * the Board shall keep complete accounts of its cost of generation, transmission, and distribution of electric energy."

Although the allocations made thus far have not as yet been used in the actual setting of power rates, tentative allocations were used by the engineers of the Joint Investigating Committee, appointed by Congress, in their study of the power rates actually in effect. They found that these rates would produce revenues sufficient to retire more than all of the cost of generation, transmission, and distribution of power, including interest. In fact, without such an allocation of costs, it would have been nearly impossible for this study to have been made.[2]

The various theories of joint-cost allocation considered by the TVA Financial Policy Committee have been previously described herein. Arguments for and against such theories are of academic interest, since the real difficulties arise in their application.

Such difficulties constitute the real objection to the direct use of the benefit theory, so-called. This method appeals to engineers as being reasonable and equitable. It has long been used in assessing costs of irrigation, flood control, and land drainage, where the costs of the projects themselves are paid for by taxation on the real property served or protected.

A little reflection will convince any experienced engineer that the value of the benefits from the various components of the Authority's program cannot be estimated with sufficient accuracy to serve as the basis for accounting procedures. These benefits are very real and very considerable, but by their very nature include a wide variety of both tangible and intangible items that cannot be evaluated with any pretense of accuracy.

There is no real justification for basing the allocation of costs for accounting purposes upon any one mathematical formula. The principal method adopted by the Authority serves more as an aid to judgment than as a formula for the accurate determination of definite values.

By this method the common costs are allocated in proportion to the remaining costs resulting from the subtraction of direct costs from the total estimated

[2] "Engineers' Report of the Joint Committee Investigating the Tennessee Valley Authority," S. Doc. No. 56, Pt. III, 76 Cong., 1st Session.

alternative costs. This amounts to assuming that the savings from multiple-purpose development are shared in by the various purposes in this same proportion. It is also equivalent to allocating the common cost in proportion to the cost of constructing separately those facilities which in multiple-purpose development are provided by the jointly-used portions of the project.

The exactness of such a method from the mathematical point of view depends upon the accuracy with which it is possible to estimate the cost of hypothetical "alternative" single-purpose developments. These estimates from the accounting standpoint have practically no accuracy at all, since they must depend upon broad assumptions regarding the design of such developments.

Nevertheless, these computations are a most valuable aid to the judgment in arriving at final allocations of cost, since they show the trends of different items, and the effect of the different and variable factors.

In the end, the actual allocation of costs must be an arbitrary and empirical process, and the writer can see no reason why this should not be recognized frankly. Since no one method has received any official or judicial approval, it is entirely reasonable to use any or all methods when these are appropriate and equitable.

Limits of Variation

It is pertinent to examine, at this point, the limits of variation in the final allocations as affected by the application of the theory. In the cost allocations the direct costs of the various purposes can be readily ascertained and are not subject to variations due to possible inaccuracies of estimates. The allocations based on the alternative-justifiable-expenditure theory, however, are affected by possible inaccuracies in the estimates of alternative costs. These possible inaccuracies have been held to a minimum in so far as possible in the allocations made thus far, since the costs which were used for alternatives were based largely on the costs of the multiple-purpose projects with comparatively minor alterations. Moreover, a comparatively large variation in the alternative costs would result in a minor variation in the final allocations as is demonstrated by Table 3, which shows the effect of variations made in the seven-plant allocation based on the alternative-justifiable-expenditure theory.

TABLE 3.—Relative Effect of Changes in Alternative Costs (Percentages)

Facility	Reduction in cost of one-purpose system[a]	Corresponding Change in the Allocation of Costs to Power	
		Common costs	Total costs
Navigation......	10	+1.8	+1.1
Flood control....	10	+1.1	+0.6
Power..........	10	−4.1	−2.6

[a] Other systems remain the same.

Viewing this problem in its broadest sense, it is evident that no allocation can be determined with mathematical precision. On the other hand, there are reasonable limits to the variations in allocations of common cost.

Where three major purposes are involved, it would be an obvious absurdity to charge more than half of the common cost to power alone, especially where power production is designated by congressional legislation as incidental to the

other two purposes. On the other hand, where power is the only "paying partner," it is difficult to justify charging to power less than one third of the common costs in any event.

The TVA allocation of common costs to power has been set at 40% for the three-plant to seven-plant systems, resulting in an average allocation to power of the total costs of about 52%. Were the allocation of common costs increased from 40% to 50%, this would increase the total to only 58% whereas a decrease in allocation of the common costs from 40% to $33\frac{1}{3}$% would reduce the total to only 48%.

The extreme reasonable range of the total allocation to power, regardless of the method adopted, therefore, is from 48% to 58%. Considering the difficulty of the problem and the wide divergence of opinion expressed in this connection, this is distinctly reassuring.

Other Similar Allocations

Bonneville Project.—Reference 6, in the Appendix, contains material quoted from the Federal Power Commission's release No. 380 with reference to the allocation of costs for the Bonneville project.

Boulder Power Plant.—The Swing-Johnson Bill, which provided for construction of Boulder project, required that before construction of the dam was started contracts should be engaged for sale of power at a price sufficient to repay the entire cost of the construction of the dam, together with interest at 4%, in a 50-yr period. The actual contracts as entered into with the various lessees provided that each shall pay a certain price for use of falling water at a dam, and that the cost of the power installation shall also be paid by these lessees at a rate sufficient to repay the cost of the power plant in 25 years, together with interest. These contracts are now in force and apparently the cost of the dam will be repaid as provided in the Act.

However, the City of Los Angeles, Calif., has endeavored to obtain a modification of the prices paid for power whereby a substantial allocation would be made to flood control, the cost of power equipment would be repaid in 50 years instead of 25, and the rate of interest on the investment would be reduced from 4% to 3%. This bill was passed by Congress in July, 1940.

Conclusions

The TVA allocation of costs may be summarized as follows:

1. The allocation was made as directed by Act of Congress;

2. Allocation is not based exactly on any one mathematical calculation or formula, the final sums being fixed by judgment and not by computation;

3. Items used solely for a particular purpose are charged to that purpose alone; it is therefore only the remaining "common costs" which require allocation;

4. A considerable variation in the allocation of common costs affects, to a much lesser degree, the proportions of the total cost charged to the three purposes; and

5. Because only the allocation of common costs is at all open to question, the possible variation in the totals due to different methods of allocation is

comparatively small. It can be shown, therefore, that the final allocations are approximately correct, within reasonable limits, regardless of the method adopted.

Most of the material used in the preparation of this paper, including estimates and similar data, was contributed by W. L. Voorduin, Assoc. M. Am. Soc. C. E., of the TVA engineering staff.

APPENDIX

Document Citations and References

Ref. 1.—Tennessee Valley Authority Act:

"Sec. 9*a*. The board is hereby directed in the operation of any dam or reservoir in its possession and control to regulate the stream flow primarily for the purposes of promoting navigation and controlling floods. So far as may be consistent with such purposes, the board is authorized to provide and operate facilities for the generation of electric energy at any such dam for the use of the Corporation and for the use of the United States or any agency thereof; and the board is further authorized, whenever an opportunity is afforded, to provide and operate facilities for the generation of electric energy in order to avoid the waste of water power, to transmit and market such power as in this act provided, and thereby, so far as may be practicable, to assist in liquidating the cost or aid in the maintenance of the projects of the Authority.

"Sec. 14. The board shall make a thorough investigation as to the present value of Dam Numbered 2, and the steam plants at nitrate plant numbered 1, and nitrate plant numbered 2, and as to the cost of Cove Creek Dam, for the purpose of ascertaining how much of the value or the cost of said properties shall be allocated and charged up to (1) flood control, (2) navigation, (3) fertilizer, (4) national defense, and (5) the development of power. The findings thus made by the board, when approved by the President of the United States, shall be final, and such findings shall thereafter be used in all allocations of value for the purpose of keeping the book value of said properties. In like manner, the cost and book value of any dams, steam plants, or other similar improvements hereafter constructed and turned over to said board for the purpose of control and management shall be ascertained and allocated.

"The board shall, on or before January 1, 1937, file with Congress a statement of its allocation of the value of such properties turned over to said Board and which have been completed prior to the end of the preceding fiscal year, and shall thereafter in its annual report to Congress file a statement of its allocation of the value of such properties as have been completed during the preceding fiscal year.

"For the purpose of accumulating data useful to the Congress in the formulation of legislative policy in matters relating to the generation, transmission, and distribution of electric energy and the production of chemicals necessary to national defense and useful in agriculture, and to the Federal Power Com-

mission and other Federal and State agencies, and to the public, the Board shall keep complete accounts of its costs of generation, transmission, and distribution of electric energy and shall keep a complete account of the total cost of generating and transmission facilities constructed or otherwise acquired by the Corporation, and of producing such chemicals, and a description of the major components of such costs according to such uniform system of accounting for public utilities as the Federal Power Commission has, and if it have none, then it is hereby empowered and directed to prescribe such uniform system of accounting, together with records of such other physical data and operating statistics of the Authority as may be helpful in determining the actual cost and value of services, and the practices, methods, facilities, equipment, appliances, and standards and sizes, types, location, and geographical and economic integration of plants and systems best suited to promote the public interest, efficiency, and the wider and more economical use of electric energy. Such data shall be reported to the Congress by the board from time to time with appropriate analyses and recommendations, and, so far as practicable, shall be made available to the Federal Power Commission and other Federal and State agencies which may be concerned with the administration of legislation relating to the generation, transmission, or distribution of electric energy and chemicals useful to agriculture. It is hereby declared to be the policy of this Act, that, in order, as soon as practicable, to make the power projects self-supporting and self-liquidating, the surplus power shall be sold at rates which, in the opinion of the board, when applied to the normal capacity of the Authority's power facilities, will produce gross revenues in excess of the cost of production of said power and in addition to the statement of the cost of power at each power station as required by section 9 (*a*) of the 'Tennessee Valley Act of 1933,' the board shall file with each annual report a statement of the total cost of all power generated by it at all power stations during each year, the average cost of such power per kilowatt-hour, the rates at which sold, and to whom sold, and copies of all contracts for the sale of power."

Ref. 2.—"Rules of Practice and Regulations" of the Federal Power Commission (dated June 1, 1938), Section 11.22:

"Reasonable annual charges for recompensing the United States for the use of Government dams or other structures owned by the United States and for the use, occupancy, and enjoyment of the lands of the United States adjoining and pertaining thereto will be based upon the estimated value for power purposes of the properties and privileges for which a license is issued * * *."

Some examples of power facilities built by private companies subject to this provision are:

Company	River	Development
Henry Ford and Son, Inc.	Hudson	Green Island
Louisville Gas and Electric Company	Ohio	Dam No. 41
Ford Motor Company	Mississippi	Twin Cities
Kanawha Valley Power Company	Kanawha	Marnet, London, and Dam No. 1
Kentucky Utilities Company	Kentucky	Lock No. 7

Ref. 3.—The Third Annual Report of the Federal Power Commission (for the fiscal year ended June 30, 1923) states regarding Project No. 362 (Twin Cities Development), Section 7, p. 256:

"To recompense the United States for the use of said dam and appurtenant structures and lands adjoining and pertaining thereto, the license to pay as a reasonable annual charge the sum of $95,440 said sum being interest * * * and depreciation * * * upon $1,193,000 an amount deemed to be so much of the cost of said dam, structures, and lands as would be justified if built or purchased solely for purposes of power development."

Ref. 4.—H. Doc. No. 709, pp. 19–20:

"The concept of alternative cost is premised on the theory that the expenditure in single-use projects would be justified by the benefits obtainable; hence, the alternative cost may be taken as a measure of the investment which the individual purposes would be justified in expending in a joint venture. Where such a cost would not have been justified, it is necessary to substitute a lower estimate based on evaluation of the benefits.

"Through the medium of alternative costs, a relative measure of value applicable to all purposes is utilized. In applying this method it is necessary to secure estimates of the lowest alternative cost by means of which substantially the same quantity and quality of service for each separate function can be obtained. The fundamental assumptions which underlie the cost estimates for single-use structures must be as reasonable and practical as they can be made and must be based upon experience and after adequate investigation."

Ref. 5.—H. Doc. No. 709, p. 20:

"After a review and extensive exploration of all the relevant methods, the committee has reached the conclusion that no one method furnishes the best general basis upon which to proceed to a final allocation that would reflect the various shares of the total joint costs to be imposed on the several functions of the Wilson, Norris, and Wheeler projects. The committee recognizes that each of the relevant methods has a measure of validity under some circumstances but that there are objections to or difficulties in the application of all the methods to the Authority's projects. Much time and effort were devoted to testing out the various hypotheses involved on the existing and proposed projects of the Authority. The allocations finally recommended are admittedly based on an exercise of judgment after considering all facts; nevertheless the committee is unanimous in believing that they furnish a fair basis for determining the relative shares of the various functions in the joint costs."

With respect to the three-plant allocation, the committee further decided that:

"The three completed projects represent a great national-defense asset, but because they are not in operation for that purpose during peacetime, it appears to be impracticable to attempt to allocate any portion of the investment to their wartime use, and the committee recommends that no part of the investment be assigned to that function. It is further recommended that no part of the joint investment be allocated to fertilizer, since fertilizer operations are

being currently charged for services rendered by other departments of the Authority."

Ref. 6.—Release No. 380, Federal Power Commission:

"The initial power development at Bonneville, which will have installed electric generating capacity of 86,400 kilowatts or approximately one-fifth of the probable ultimate installation, is slated for completion by June 30, 1938, at a total estimated cost, including interest during construction, of $9,180,500 for facilities solely for power purposes. The Commission has allocated this entire sum to initial power development and, in addition, has allocated $2,501,900 of the total estimated cost to June 30, 1938, of facilities having joint value for the purposes of navigation and power development.

"The $2,501,900 figure was obtained through the determination that, ultimately, power development may fairly bear 32.5 percent of the cost of facilities having joint value for the production of electric energy and other purposes. The total estimated cost of such facilities to June 30, 1938, including fishways, is $38,490,700 and the present allocation to initial power development is one-fifth of 32.5 percent of this sum.

"The total cost of the Bonneville project when completed is estimated at $74,144,600. In setting this figure, the Commission has estimated the cost of ultimate power development with ten generating units having an aggregate capacity of 504,000 kilowatts, together with the cost of all at-site appurtenant facilities for power use only, to be $29,448,000; and the cost of facilities having joint value for navigation and power purposes, including fishways, to be $39,179,000. It is estimated that the cost of facilities for navigation purposes only will be $5,517,600 as of June 30, 1938, and no additional capital costs for facilities solely for navigation purposes are contemplated.

"On the basis of the Commission's present determination that ultimately power development may fairly bear 32.5 percent of the cost of facilities having joint value for the production of electric energy and other purposes, $42,181,000 or approximately 57 percent, of the total ultimate cost of the Bonneville project would be allocated to power development."

AMERICAN SOCIETY OF CIVIL ENGINEERS

Founded November 5, 1852

PAPERS

DEVELOPMENT OF TRANSPORTATION IN THE UNITED STATES

By J. E. Teal,[1] M. Am. Soc. C. E.

Synopsis

The first railroad chartered in the United States was the Baltimore and Ohio, in 1828, and the story of the development of the railroads that followed, which made possible the United States as it exists today, is intensely interesting. Other forms of transportation have come into being, and some have grown to a point where they are having a decided influence on the social and economic environments of every community in the Nation. These agencies of transportation, under present conditions, are all competitive with each other, and certain economic influences have developed that apparently have resulted in unfair competition between them.

With the importance of the issues in mind, the writer has endeavored, in this paper, to sketch the history of transportation during the past one hundred or more years, to outline briefly some of the economic conditions, and to discuss a few of the pertinent factors involved. An effort has been made to correlate facts and data pertaining to the subject, which is now a transportation problem so vital and so far-reaching that a sound national economy may depend on its solution.

Railway Transportation

During the past 113 years, railway development in the United States passed through several distinct stages. The "First Stone" of the Baltimore and Ohio Railroad was laid on July 4, 1828, at which ceremony the venerable Charles Carroll, of Carrollton (Va.), remarked:

> "I consider this among the most important acts of my life, second only to the signing of the Declaration of Independence, even if indeed it be second to that."

From that date to 1850 was a period of exploration, both in the matter of

Note.—Written comments are invited for immediate publication; to insure publication the last discussion should be submitted by **April, 1942.**

[1] Transportation Engr., C. & O. Ry., Richmond, Va.

developing road, equipment, and plant facilities, and in the methods of finance and management.

The next forty years was a period of very rapid railway expansion. The railway systems had been pushed west across the Allegheny Mountains to Chicago, Ill., and the Union Pacific and the Central Pacific railroads had been completed by 1869. Railway development was rapid during the period leading to the depression of 1873. The decade 1880 to 1890 witnessed an increase in railway lines aggregating more than 70,000 miles, when a number of transcontinental lines were constructed.

Fig. 1.—Railroad Development Was Rapid During the Period Leading to the Depression of 1873

During this period of rapid railway construction, public authorities and federal, state, and local governments became convinced that the railways were indispensable to the prosperity of the Nation. Railway construction was encouraged by means of federal land grants and financial subsidies until about 1871. Some financial aid was contributed after that year.

From 1890 to the period of federal control (1917 to 1920) might be considered as the finishing period of the railway plant. Railway construction consisted mainly of the building of feeder and cross lines or new branch lines—the rounding out of the railway system as it is known today. Railway mileage was increased from 163,597 in 1890 to the peak of 254,251 in 1916, but this had receded to 235,064 miles in 1939. Track mileage reached a maximum of 429,883 miles in 1930, declining to 408,350 miles in 1939; thousands of miles of branch and feeder lines have become obsolete.

The last period, covering two decades, is of great significance in the railway picture; it begins when highway transportation entered upon a new era of development. As the result of competition growing out of common-carrier service on the highways, the revival of waterways, the expansion of pipe-line transportation, and the coming of the airplane, the railway industry has had to "tighten its belt" in order to meet financial obligations.

Railroad Operating Results Since 1920

The total property investment in Class I railways amounted to \$26,527,000,000 in 1930, decreasing to \$26,131,000,000 at the end of 1939. Peak revenues reached the total of \$6,383,000,000 in 1926, declined to an average of \$3,622,000,000 per yr from 1931 to 1937, and were \$3,995,000,000 in 1939, a 13-yr decrease of 37%.

Net income, after fixed charges, reached its peak in 1929, with a total of \$897,000,000, but only averaged \$35,000,000 per annum from 1931 to 1937; in 1938 there was a net deficit of \$129,000,000, with a net gain of only \$93,000,000 in 1939. Railway taxes increased from \$276,000,000 in 1921 to \$356,000,000 in 1939, or 29%; and taxes per one dollar of revenue increased from 5.0¢ in 1921 to 8.9¢ in 1939, or 78%. The rate of return earned by the railroads on their investment in transportation property averaged 4.81% in 1929, 1.67% for the years 1931 to 1937, 1.43% in 1938, and 2.26% in 1939, a 10-yr decrease of 53%.

Railroad freight traffic reached 447,000,000,000 ton-miles, its peak, in 1929, averaged 292,000,000,000 ton-miles from 1931 to 1937, and 333,438,000,000 ton-miles in 1939, a 10-yr decrease of 25%. Railroad passenger traffic reached its peak in 1920, when 46,800,000,000 passenger-miles were handled, declined to an average of 19,900,000,000 from 1931 to 1937, and amounted to 22,651,000,000 in 1939, a 19-yr decrease of 52%.

Railroad freight rates declined from 12.75 mills per ton-mile in 1921 to an average of 9.96 mills from 1931 to 1937, and 9.74 mills per ton-mile in 1939, an 18-yr decrease of 23%. Average revenue per passenger-mile declined from a maximum of 3.86¢ in 1921 to 2.033¢ from 1931 to 1937, and 1.839¢ in 1939, an 18-yr decrease of 52%. Competition has been an important factor in the downward trend in railroad freight and passenger rates.

Railroad employment declined from the peak figure of 2,022,000 in 1920 to an average of 1,063,000 from 1931 to 1937, and 988,000 in 1939, a 19-yr decrease of 51%.

Because of the failure of many carriers to meet their interest charges, the credit of the railroad industry in general has been greatly impaired, and must be recouped in the future. The meager financial results of railroad operations for 1938, amounting to 1.43% return on the investment, were secured by heavy reduction in maintenance and other expenditures, some of which must be made up at some future date. Closely related to the subject of maintenance is the decrease in ability of the railroads to contribute to the national economy in the form of purchases of the products of manufacturers and other industry.

The gross capital expenditures made by the railroads for the period 1921 to 1930 were an average of \$773,000,000 per annum. For the period 1931 to

1939 this average amounted to only $262,000,000, or a decrease of $511,000,000, or 66%, per annum. Comparing the purchases of fuel, material, and supplies during the same periods, the average annual expenditure was reduced from $1,384,000,000 for the first period to $660,000,000 for the second period, a decrease of $724,000,000, or 52% per annum. These statistics show that under normal conditions the railroads are heavy consumers of the products of other industries and are, therefore, an important factor in the Nation's business. Also, the capability of the railroads to buy equipment and other supplies, and to foster general employment, has been seriously impaired in recent years.

More than 75,000 miles of railroad lines, or about one third of all railroad mileage, are now in the hands of receivers or trustees. This is the largest proportion of rail mileage in bankruptcy ever recorded in the history of the United States.

It is only necessary to note the many improvements that have developed in rail transportation during the two decades 1920–1940, and more particularly, the latter of the two. Every one knows of the modern, streamlined, air-conditioned, high-speed passenger trains, and the faster freight, including the pickup and delivery services.

Inland Waterway Transportation

The history of inland waterway transportation, of course, antedates that of railroad transportation. Rivers and canals were among the principal channels of transportation in the United States until the close of the Civil War. In Virginia, the James River Company, organized by George Washington and others, as a stock company, in 1785, had as its object the construction of a canal from the tidal basin of the James River at Richmond, Va., following that river to Clifton Forge, Va., and across the Allegheny Mountains, by highway, to the Kanawha River, a tributary of the Ohio River in West Virginia. The canal was completed for a distance of 200 miles, when financial troubles stopped construction. The property was later taken over by the predecessor of The Chesapeake and Ohio Railway Company. By the close of 1825, the Erie Canal had been completed between the Hudson River and Lake Erie, 351 miles, and in 1828 the Chesapeake and Ohio Canal was under construction from the tidal basin of the Potomac River at Washington, D. C., for a distance of 186 miles, following the Potomac River to Cumberland, Md.

Canal construction spread very rapidly, not only in those states along the eastern seaboard that were in need of lines of communication with the Middle West, but also within the Western States. Following the Civil War, in 1865, the development of railways attracted much of the commerce that formerly moved by water. The railways also created new commerce in many areas not served by rivers, following which traffic on the rivers and canals declined to a condition of little importance.

The first turnpike roads were built in the United States with the view that the full cost of transportation should be borne by the shipper. Likewise, it was the intention to charge tolls for the use of the canals; but returns from this source rarely were sufficient to cover both capital charges and maintenance expenses. The Pennsylvania canals were a complete financial failure. The

New York system of inland waterways, including the Hudson River and the Erie-Champlain canal, made a better showing, but had to be (and still are) supported substantially by general taxation. The total cost of canals to the people of New York from the beginning to 1929 has amounted to more then \$346,000,000. From 1930 to 1940, inclusive, the total expense aggregated \$109,195,000, or an average of \$9,927,000 per annum, divided thus: \$31,398,000, or \$2,854,000 per annum, for operation and maintenance, and \$77,797,000, or \$7,073,000 per annum, for carrying charges on construction cost, computed at 4% per annum. This brings the grand total of expenditures, to 1940, to \$455,195,000. Annual credit for revenue received by the commissioner of canals and waterways for various services amounted to \$607,000 for 1940.

By 1885, the rivers and canals had lost most of the high-grade traffic. However, low-grade traffic, such as sand, gravel, and coal, continued to move in fairly large quantities for a longer period. By the end of the century, with the exception of coal traffic on the Monongahela and Ohio rivers, canal and river transportation was unimportant.

Fig. 2.—The Inland Waterways Renaissance Received a Great Stimulus During the First World War

The inland waterways renaissance, which began in the last decade of the nineteenth century, received a great stimulus during the first World War, and since the war has been carried forward with the assistance of fostering legislation and the spur of many government officials, individuals, and organizations. To bring this question down to date, it suffices to state that the report by the

chief of engineers of the United States Army for 1939 shows that a total of $3,637,528,923 has been spent or authorized on the construction and maintenance of waterways projects to June 30, 1939, of which approximately $2,655,000,000 has been spent or authorized for river and harbor navigation projects. Of this amount, approximately $1,123,000,000 was on the Mississippi River system, $517,000,000 for other waterways, and the remainder for seacoast and lake harbors and channels and intercoastal canals. The total mileage of inland waterways, according to the Board of Engineers for Rivers and Harbors, as of June 15, 1935, aggregated 26,406, divided as follows:

Inland waterways	Route mileage
Atlantic Coast rivers	5,263
Gulf Coast rivers, including the Mississippi River system	17,381
Pacific Coast rivers	1,796
Intercoastal waterways	1,966
Total	26,406

Of the total mileage, 15,688 miles have a depth of less than 6 ft; 5,394 miles have depths ranging between 6 ft and 9 ft; 4,746 miles have depths ranging from 9 ft to 12 ft; and 1,578 miles have depths greater than 12 ft.

TABLE 1.—WATER-BORNE TONNAGE

Year	Short Tons (Thousands)		
	Great Lakes	Other	Total
1920	98,751	125,400	224,151
1925	113,644	204,569	318,213
1930	115,727	226,760	342,487
1935	89,356	225,917	315,273
1936	122,788	276,264	399,052
1937	143,356	313,287	456,643
1938	82,480	277,755	360,235

The total tonnage of water-borne commerce on the Great Lakes, and on rivers, canals, and connecting channels in the United States, after eliminating the known duplications, is shown in Table 1 for the calendar years 1920 through 1938. From this table it will be seen that the tonnage other than on the Great Lakes has increased from 125,400,000 tons in 1920 to 277,755,000 tons in 1938, an increase of 121%.

The total ton-mileage on inland waterways of the United States during 1938, exclusive of the Great Lakes, was 17,742,506,538. The total ton-mileage on the Great Lakes was 49,004,019,901, which, together with that on the rivers, canals, and connecting channels, makes a total ton-mileage of 66,746,526,439. Computed on the basis of 4.46 mills per ton-mile (average for Inland Waterway Corporation 1924 to 1935), the expense to the shippers using rivers, canals, and connecting channels for transporting 17,742 million tons one mile in 1938 would be approximately $79,000,000. The cost of owning and maintaining the waterways, of course, is borne by the federal government.

Highway Transportation

The development of highways in the United States has passed through three periods, each characterized by usage of a distinct character. In the first period, extending to about the middle of the nineteenth century, the initiative

in furnishing major highway routes was taken by the federal government. This was gradually relinquished, until about 1856 all federal jurisdiction over the national highway routes had been assumed by the states through which the routes passed. During this time a number of states had found the development of subsidiary highway routes to be burdensome and had placed some of them on a commercial basis by granting charters to individuals or companies, permitting them to build and operate toll roads. The states, however, participated in the cost of these projects and retained the right to regulate them.

Prior to 1892 the development in the field of highway transportation involved no particular complex problems of public policy. Both the building of the roads and the building of the motor vehicles were passing through experimental phases. The use of the public highways for purely commercial purposes had not attained sufficient importance to cause any concern on the part of the established carriers.

The second period in the development of the highway transportation as it exists today embraces the one of rapid railroad building, extending to about 1900. As the railroad mileage expanded, the use and necessity of the highways decreased to a position of minor importance. As transportation over public roads became unnecessary and unprofitable except for short distances, the building and maintenance of these roads were reduced to a matter of local importance only and consequently were left to the small political subdivisions, such as the county, township, and road district. It follows that from the middle to the end of the nineteenth century only a small proportion of highway travel passed beyond the boundaries of local political subdivisions, and the administration and financial responsibility for furnishing highway facilities thus narrowed down to such areas.

The year 1900 marks the beginning of the third period, which is characterized by a very rapid increase in the use of the highways. As the automobile developed, it soon became apparent that transportation on the highways for longer distances was possible, resulting in the creation of traffic that bore little or no relation in point of origin and destination to the political areas through which it passed. Such traffic was of an entirely different character than the type that moved wholly within the local subdivisions. It was also realized that the cost of highways required by these different types of traffic could not with equity be assessed entirely against the local property owners, but that a certain part of the cost should be borne by the taxpayers at large.

From 1900 to 1920 the development of the highway was somewhat similar to the evolution of the automobile—that is, as it developed, it passed through various experimental stages as to types of materials used in its construction. As the volume of traffic grew, the wear and tear on the highways became more apparent, and better and stronger materials were required to maintain them.

During the two decades (1920 to 1940), however, rapid expansion of improved highways occurred, and the perfecting of the motor-vehicle equipment has given rise to a substantial volume of commercial operations on the highways. The competition therein afforded by highway transport has been

the cause of grave concern, particularly to the railroads, and it has seriously affected their revenues.

In 1921 the highway mileage of the United States aggregated 2,924,505, of which 202,915 miles, or 6.9%, were under the control of the state authorities. The total mileage increased to 3,009,066 in 1930, of which 324,496 miles, or 10.8%, were under the jurisdiction of state authorities. Only 84,372 miles, or 41.6%, of the state-controlled mileage in 1921 were surfaced, and, of this mileage, only 14,707 miles, or 17.4%, were of the high-type surface.

Fig. 3.—The Competition Afforded by Highway Transport Has Been the Cause of Grave Concern to the Railroads

By the close of 1931, significant shifts had occurred in this distribution. The rural highway mileage had expanded only 110,388 miles, or 3.8%, whereas the state highway system had added 126,027 miles, an increase of 62.1%. About 12% of this increase in state mileage was caused by the transfer of local mileage to state systems, rather than by an expansion of the entire road mileage. It is also significant that by the end of 1931 about 74% of the state highway system mileage was surfaced, as contrasted with 42% in 1921. More than 33% of the mileage in 1931 had high-type surface, as compared with 17% in 1921.

During 1931 and subsequent years, various state highway commissions have taken over the supervision of the construction and maintenance of certain county highways which, together with some road mileage previously reported, formed a secondary system of state highways; and at the close of 1938 the

existing mileage of public roads and streets under state control aggregated 541,841 miles, distributed as follows:

United States highway system (1938)	Miles
Primary state highway system	326,852
Secondary state highways and county roads under state control	194,428
Total	521,280
Urban expansion of state highways	23,804
Total mileage under state control	541,841

Of this mileage, 424,791 miles, or 78%, have been surfaced. From the foregoing data it may be seen that the state-controlled mileage increased from 202,915 in 1921 to 328,942 miles in 1931, or 62%, and to 541,841 miles in 1938, or 65% over 1931, and 167% over 1921.

Estimates[2] indicate the total cost of the highway system at the close of 1939, including state highways, county roads, and city streets, to be $35,218,000,000, divided as shown in Table 2.

TABLE 2.—HIGHWAY COSTS (MILLION DOLLARS); 1939

Item	Period	State highways	County highways	City streets
1	Prior to 1921	741	4,276	1,921
2	1921 to 1932[a]	4,008	4,720	5,210
3	1933 to 1937[a]	3,933	3,042	3,269
4	1938 to 1939[a]	1,573	1,216	1,308
	Totals	10,255	13,254	11,708
		Grand total		35,217

[a] Inclusive.

The Bureau of Statistics, Interstate Commerce Commission, placed the investment in highways and vehicles used in intercity transportation in the United States at $24,232,000,000, as of 1937. Approximately $12,187,312,000 has been collected from the users of streets and highways through gasoline taxes, the amount for 1939 being $821,656,000 net.

The first automobile appeared in 1892, and from that time the growth of automotive transportation and the development of the extensive highway system in the United States are coincident and interdependent. The record indicates that in 1895 there were four registered motor vehicles. This number increased to 8,000 in 1900, and to 55,000 in 1904. Of these 55,000 vehicles, 410 were classified as trucks and road tractors. From this date, the number of automobiles classified as passenger cars, taxis, buses, trucks, and road tractors increased annually to a total of 9,231,941 in 1920, divided thus: 8,225,859 passenger vehicles and 1,006,082 trucks.

A peak of all motor-vehicle registrations was reached in 1930, when a total of 26,545,281 vehicles was recorded, of which 86.9% were passenger vehicles and 13.1% were trucks. During the next few depression years, the number of registrations decreased, and totaled 23,827,290 in 1933. The number in 1940 was 32,452,861, of which 27,434,979 were privately owned passenger

[2] "What Is Public Aid to Transportation?" produced and distributed by the Assn. of Am. Railroads. Estimates for periods up to 1937, inclusive, by C. B. Breed, Clifford Older, and W. S. Downs, Cons. Engrs. Estimates for 1938 and 1939 based on previous 5-yr period.

vehicles, 4,590,386 were trucks, and 427,496 were publicly owned vehicles. The percentage of motor trucks had increased from 1.7 in 1904 to 14.2 in 1940.

Passenger-car traffic is fairly homogeneous as to both character of traffic and type of equipment; no such uniformity, however, exists in truck operations. Widely different types of vehicles are used in the several kinds of truck operations, and variations in ownership and operation exist and various types of services are rendered.

The number of trucks engaged in short commercial or private hauling over the highways is not accurately known. It has been estimated, however, that, of the 4,590,000 privately owned motor and tractor trucks, approximately 1,307,000 are farm owned, 2,626,000 are privately owned and not operated for hire, and of the remaining, approximately 262,000 are common carriers and 395,000 are contract carriers. Approximately 86% of all the trucks are operated by the owners, and about 14% are common and contract carriers, operated for hire, or under hauling agreements.

TABLE 3.—Motor-Vehicle Statistics; 1940, 1939, 1938.

No.	Description	1940	1939	1938
	(*a*) Income (Thousands of Dollars)			
1	State gasoline tax collections—net[a]	\$870,692	\$822,013	\$772,060
2	Registration fees for all vehicles	373,771	353,533	340,061
3	Fees for permits, certificates of title, fines and penalties	65,407	58,961	48,764
4	Total, items 1 to 3[b]	\$1,309,870	\$1,234,507	\$1,160,885
	(*b*) Number of Motor Vehicles Registered, and Gasoline Consumed (Thousands)			
	Number of Motor Vehicles Registered:[c]			
5	Private and commercial vehicles	32,026	30,615	29,486
6	Owned by federal government	143	121	110
7	Owned by state, county, and municipal governments	284	274	257
8	Total	32,453	31,010	29,853
9	Gasoline consumed (gallons)	21,913,000	20,638,000	19,507.000
	(*c*) Units			
10	Average gaoline tax (cents) per gallon[d]	\$3.97	\$ 3.98	\$ 3.96
11	Average persons per vehicle in the United States[e]	4.1	4.3	4.4
12	Average car registration fee in the United States[f]	\$11.67	\$11.55	\$11.53

[a] Plus inspection fees, dealers' license fees, fines and penalties, and aviation gasoline.
[b] Motor-vehicle users in all states paid some or all of the taxes and fees included under items 1 to 3.
[c] Includes trucks: 4,590,386 in 1940, 4,320,829 in 1939, and 4,224,031 in 1938 (see Table 4).
[d] Item 1 divided by item 9. Rates for individual states range from 2¢ to 7¢ per gal.
[e] Population divided by item 5. 10.4 persons in 1921.
[f] Item 2 divided by item 5.

Truck operations have been adapted to a wide range of transportation tasks, which vary from the handling of light farm produce and transportation of household goods, merchandise, and machinery, to the heavy services of logging camps and construction work. The distances over which these operations occur vary widely. Common carriers and contract trucking operations are scheduled in runs of from 50 to 100 miles to more than 1,500 miles.

Small-scale operations and individual managements were characteristic features of commercial trucking operations until about 1930. Since that time there has been a tendency to consolidate trucking operations throughout the country.

Under provision of Act of Congress dated August 9, 1935, the common-carrier truck operators and contract operators handling interstate commerce are subject to the jurisdiction of the Interstate Commerce Commission, and for the period closing with March, 1940, the Commission received reports of revenues, expenses, and statistics representing 1,106 motor carriers of property.

Table 3 shows motor-vehicle statistics for 1938, 1939, and 1940, compiled annually by Federal Works Agency, Public Roads Administration. Registered motor trucks by capacity groups as of December 31, 1939, were as shown in Table 4.

TABLE 4.—REGISTERED MOTOR TRUCKS AS OF DECEMBER 31, 1939

Capacity (Tons)		No.	%
From	To[a]		
....	>1.0	1,641,166	38.18
1.0	1.5	84,974	1.98
1.5	2.0	2,247,219	52.27
2.0	2.5	132,935	3.09
2.5	3.5	108,490	2.52
3.5	5.0	44,138	1.03
<5.0	....	40,078	0.93
Total...........		4,299,000	100.00

[a] To, but not including.

Information is not readily available covering the financial status of the various trucking concerns, except through the reports to the Interstate Commerce Commission, which show that for 1939 a total of 1,105 carriers were reported as receiving gross operating revenues aggregating $425,373,099, and operating revenues of $20,663,530, divided thus: 82.5% by common carriers and 17.5% by contract carriers.

For the same year (1939), 196 motor carriers of passengers reported revenue aggregating $143,108,224, of which $123,899,080 was for passengers carried between cities and $19,209,144 was received from local or suburban passengers. The net revenue reported amounted to $20,249,003.

Oil and Gasoline Pipe Lines

Pipe lines serve the oil industry by: (1) Assembling crude oil at central points for storage or shipment to refineries; (2) shipments of crude oil to refineries or marine shipping terminals by trunk lines; and (3) long-distance shipment of the principal refined products from field or tidewater refineries or tank terminals. Pipe lines are classified as gathering and trunk lines. The gathering lines are usually laid on top of the ground, and range in size to about 4 in. in diameter, whereas the trunk lines are laid underground, below freezing, and are commonly 6 in. to 8 in. or more in diameter.

The pipe-line mileage of companies reporting to the Interstate Commerce Commission is equal to about 42% of the railway main-track mileage, and amounted to 98,661 miles in 1939, divided thus: 59,108 miles in trunk lines and 39,573 miles in gathering lines.

The development of pipe lines marched side by side with the development of the oil industry. The first successful line, 4 miles long, began operation in

the oil fields of western Pennsylvania, in December, 1865, and, 10 years later, a 4-in. line, 60 miles long, was laid to Pittsburgh, Pa. In 1892, there were about 3,000 miles of pipe lines in service, mostly 6 in. in diameter. These connected the oil fields of western Pennsylvania with the refineries in the Eastern States. About 96% of the oil produced prior to 1900 was from fields east of the Mississippi River. During the next 30 years this relationship changed and, by 1930, 66% of the oil came from west of the Mississippi River. In that year the states of California, Arkansas, Oklahoma, and Texas, together, produced more than 86% of all the crude oil in the United States.

Between 1892 and 1914 there was great expansion in pipe-line construction, and at the end of that period five major companies operating in the mid-continent field had a total of 6,059 miles of trunk lines and 3,956 miles of gathering lines. This expansion extended into California, and five companies operating there had lines aggregating 2,439 miles in 1919.

Since 1920 the development has been part of the growth of large oil companies. The numerous oil discoveries in the two decades 1920 to 1940 have resulted in adequate oil reserves and the growth of industry in its many phases; the automotive uses for oil and its products, especially, has increased greatly the demands on the oil industry.

Until 1930, pipe lines were used mostly for transporting crude oil to refineries, and refined oil and gasoline to shipping terminals; since that time they also have been utilized in transporting gasoline from the refineries and shipping terminals to market centers. This practice now exists along the eastern seaboard and from mid-continent fields to Lake Michigan and to Ohio, upper Mississippi, and Missouri river valley points.

The transportation of oil and oil products through pipe lines has become a major industry in itself. In 1921, there was a total of 55,260 miles of gathering and trunk pipe lines operated by companies reporting to the Interstate Commerce Commission. This mileage gradually increased to 98,681 miles in 1939. During the same period, the investment in the industry increased from \$365,000,000 to \$829,646,000 whereas the operating revenue increased from \$115,900,000 to \$212,466,000. For 1939, the operating expenses were \$97,130,000 and taxes were \$31,936,000, leaving a net of \$83,401,000. (Fig. 4 shows several hundred of the 20,000 surplus tank cars used in 1941 to alleviate the oil shortage on the East Coast due to the National Defense situation.)

The interstate pipe lines became common carriers and were placed under the jurisdiction of the Interstate Commerce Commission under the Hepburn Act of June 29, 1906, since which time the authority of the Commission has extended over this industry.

The extent to which pipe-line companies compete with the railroads and inland waterway carriers is of considerable interest. Data are not readily available as to the volume of oil products now being shipped by common and contract carriers over the inland waterway routes; however, statistics compiled by the Director of the Bureau of Statistics, Interstate Commerce Commission, and published in February, 1939, show that the railway revenue from transportation of crude petroleum and petroleum products decreased from

$287,027,000 in 1928 to $187,977,000 in 1937, whereas pipe-line revenues increased from $206,438,000 in 1928 to $240,362,000 in 1937.

It is obvious that pipe-line competition has been seriously felt by the railroads. In the past it has not been possible to compare relative cost of pipe-line transportation with railroad transportation. It is the general opinion of railway cost engineers that it is impossible to determine the exact cost of rail transportation for handling oil traffic. However, it appears that, as compared with railroads, the cost of pipe-line operations is relatively low, because of the comparatively small number of employees required. On the other hand, the pipe lines are the only form of transportation other than the railways that do

FIG. 4.—THE TRANSPORTATION OF OIL HAS BECOME A MAJOR INDUSTRY

not now receive annual subsidies from the government. The pipe lines never received land grants.

AIR TRANSPORT

Aviation with heavier-than-air machines was born on the sand dunes at Kitty Hawk, N. C., on December 17, 1903, when Wilbur and Orville Wright succeeded in flying a machine equipped with planes and a small gasoline motor for a distance of 260 yd. The Wright Brothers continued their experiments, and on October 5, 1905, at Dayton, Ohio, they succeeded in making their heavier-than-air machine, powered with a gasoline motor, fly a distance of 24 miles, at a speed of 38 miles per hr. During the next 10 years many experiments were conducted in this field, and at the beginnning of the first World War belligerent nations found a practical use for flying. This period (1914 to 1917) provided a powerful stimulus to all branches of flying, and great improvements were made in technical equipment. Intensive development during this period led to constant changes not only in the practice but in the application of flying, so that, when peace came, it was expected that progress would be as

rapid as in wartime. The expected progress, however, did not seem to materialize immediately, for at the beginning of 1926 many war experiences had not been fully digested, nor their lessons properly formulated.

The Aeronautics Branch of the U. S. Department of Commerce was established in 1926, and efforts were made to provide airways and aids to air navigation. The first domestic contract air-mail routes were established in that year. Since 1926, this industry has taken the best of skill and methods from older industries, and at the same time it has developed types of materials, devices, and methods best suited to its own efficiency and requirements. Today, less than four decades from the beginning, no industry is more alive to the possibility of technical advance.

The development and production of airplanes have made possible an astonishing advance in the field of transportation, which probably has no parallel in the history of any other common carrier. It was in 1929 that scheduled air transport pioneers first began to be operative. Once each day a small single-engine airplane took off from the Newark airport (which had been recently reclaimed from the New Jersey marshes) and made its way toward the Pacific Coast. It carried an occasional passenger and some mail. Today the national air map reveals a great network of air lines, reaching from the Atlantic to the Pacific, and from the Great Lakes to the Gulf of Mexico. These lines normally fly almost twice as many miles as do the combined air lines of Great Britain, France, Germany, Italy, and the Netherlands; they transport more passengers and more mail, and they surpass the air lines of any other nation in comfort, speed, and safety. Within the same brief period of a little more than one decade, the pioneers of air commerce have linked the United States in scheduled air transportation with Hongkong, China, a distance of 8,746 miles across the Pacific Ocean; with Bermuda, 784 miles from New York in the Atlantic Ocean; and with Central and South America, 7,318 miles from Miami, Fla., to Buenos Aires, Argentina. Since July, 1939, the United States has established transatlantic scheduled air transportation with Great Britain, Ireland, France, and Portugal, 3,437 miles. This service, however, has been interrupted in so far as the belligerent nations are concerned. A measure of the progress of aviation is afforded by reference to Table 5, which contains

TABLE 5.—PROGRESS OF AERONAUTICS IN THE UNITED STATES

Year	Total length of airways (miles)	Total Freightage (Lb)		Total Miles Flown		Passengers carried
		Express	Mail	(Revenue)	Passenger	
1929	36,330	257,443	7,772,014	25,141,499	103,747,249[b]	173,405
1940	94,079	14,188,178	32,700,000[a]	119,517,263	1,265,164,059	3,185,278
Increase (%)	159	5,411	321	375	1,119	1,737

[a] Estimation, based on ton-mileage and tons handled in 1935. [b] Data for 1930.

selected statistical information.[3] It shows the progress of civil aeronautics in the United States by calendar years, beginning with 1926.

[3] "Progress of Civil Aeronautics in the United States," U. S. Dept. of Commerce, Civil Aeronautics Administration, Washington, D. C.

Table 6 shows the financial results[4] for all domestic air carriers for the fiscal year 1939 compared with 1935.

Up-to-date information is not readily available as to the private and public investment in the air transport industry. However, the Bureau of Statistics, Interstate Commerce Commission, reports a total investment for airways and aircraft of $171,000,000 in 1937.

TABLE 6.—OPERATING REVENUES AND EXPENSES OF DOMESTIC AIR CARRIERS

No.	Description	1935	1939	Increase (%)
	Operating Revenues:			
1	Other than United States mail	$14,128,249	$30,650,695	117
2	United States mail	8,814,296	16,669,197	89
3	Total	$22,942,545	$47,319,892	106
4	Operating expenses	$26,264,961	$46,326,581	76
5	Profit (P) or Loss (L)	$ 3,322,417 (L)	$ 993,311 (P)	...

The government aids airway carriers by: (1) Establishment and maintenance of air navigation facilities, including construction and maintenance of intermediate landing fields, radio broadcasting stations, radio range beacon stations, radio landing beacons and landing fields, and radio lighting facilities; (2) weather services provided by the U. S. Weather Bureau, U. S. Department of Agriculture; and (3) substantial air-mail contracts. It has been estimated that the sum total of all public aids to this transport agency on a cumulative basis to the present time is more than $400,000,000.

INVESTMENT AND TRAFFIC

The foregoing outlines briefly the history of the transportation agencies serving the people of the United States, and demonstrates that there is a large investment in each. It has been impossible to obtain comparable information on the subject, because of the different methods that have been used by the various agencies in accounting for maintenance and investment. However, the total investment in intercity transportation facilities as compiled by the Bureau of Statistics, Interstate Commerce Commission,[5] as of 1937, was $53,215,000,000 as shown in Table 7. This includes steam railways (including express and sleeping car companies), electric railways, highways and vehicles, waterways and equipment, pipe lines, and airways and aircraft.

TABLE 7.—INVESTMENT IN INTERCITY TRANSPORTATION FACILITIES IN THE UNITED STATES (1937)

Facility	Million dollars	Percentage
Steam railways, express, and sleeping car companies	23,706	44.6
Electric railways	379	0.7
Highways and vehicles	24,232	45.5
Waterways and equipment	3,782	7.1
Pipe lines	945	1.8
Airways and aircraft	171	0.3
Total	53,215	100.0

[4] "First Annual Report," Civil Aeronautics Authority, 1939.

[5] Report of Bureau of Statistics, Interstate Commerce Commission, submitted to House Committee on Interstate and Foreign Commerce, January 24, 1939; hearings on H. R. Doc. No. 2531, Vol. 1, pp. 24–25.

The volume of traffic, measured by revenue ton-miles and passenger-miles, handled by the various agencies is shown in Tables 8(*a*) and 8(*b*) for 1939 as compared with 1926. The data in these tables were compiled from the report of the "Committee of Six" appointed in 1938 by the President of the United States to study the transportation situation, corrected to date.

Study No. 3951, made by the Bureau of Statistics, Interstate Commerce Commission, showing the fluctuation in carload freight traffic compared with production for 1928 to 1938 inclusive, shows that the ratio of the actual

TABLE 8.—COMPARISON OF TRAFFIC AND RE

No.	Facility	(*a*) REVENUE TON-MILES				
		1926		1939		Change[a]
		Millions	%	Millions	%	
	(1)	(2)	(3)	(4)	(5)	(6)
1	Steam railways	447,444	75.4	335,375	61.7	25.0 D
2	Electric railways	1,313	0.2	725	0.1	44.8 D
3	Intercity motor carriers	23,530	3.9	46,000	8.5	95.5 I
4	Great Lakes	90,037	15.2	76,312	14.0	15.2 D
5	Other inland waterways	9,543	1.6	19,937	3.7	108.9 I
6	Pipe lines	21,700	3.7	65,015	12.0	199.6 I
7	Airways	....	....	11	....	....
8	Total	593,567	100.0	543,375	100.0	8.5 D

[a] D = decrease and I = increase. [b] Intercity buses. [c] Includes Pullman and express companies estimat
estimated for 1926.

railroad tons handled to the potential railroad tons declined progressively each year, with the exception of 1934, when the percentage was one point up, until only 78.7% of the potential tonnage was handled in 1938, compared with 1928.

The revenue of commercial carriers for 1939 compared with 1926 is shown in Table 8(*c*). From these tables, it will be seen that the economic position of the steam railways has been declining, and that there has been a very substantial increase in the business of the inland water, highway, and air carriers.

The changes in the transportation "picture" are very significant. What have been the causes resulting in the shift of passengers and freight from the railways to the highways, waterways, and airways? In the broad sense, they are entirely economic. Passengers and shippers use the transportation agencies offering the most satisfactory service at the least direct expense to them. The expense factor being equal, the service factor will govern. The real question before the people, therefore, will be to determine whether or not the free laws of economics govern, and, if not, to what extent competing carriers should be controlled by public authority. It will then be up to Congress to find a remedy to correct the evils and protect fairly and equitably the interests of all concerned.

Taxes Paid by Transportation Agencies

In 1939 the railways paid 8.9¢ of each dollar of revenue for taxes. This is an increase of 78% over 1921, and the trend is still upward. In addition, the railways bear the expense of owning, maintaining, and protecting their properties, amounting to 24¢, which sum, plus taxes, makes a total of 33¢ per dollar of revenue.

The pipe lines reporting to the Interstate Commerce Commission for 1939 had similar expenses, aggregating 15¢ for taxes and 23¢ for owning and main-

NUE IN THE UNITED STATES, IN 1926 AND 1939

(b) Revenue Passenger-Miles					(c) Revenue of Commercial Carriers				
1926		1939		Change[a]	1926		1939		Change[a]
Millions	%	Millions	%		Millions	%	Millions	%	
(7)	(8)	(9)	(10)	(11)	(12)	(13)	(14)	(15)	(16)
35,673	75.2	22,713	56.8	36.3 D	$6,701[c]	83.0	$4,140	67.1	38.2 D
5,537	11.7	956	2.4	82.7 D	....[d]	....	....	....	....
4,375[b]	9.2	14,147	35.4	223.4 I	475[e]	5.9	1,048	17.0	120.6 I
1,849	3.9	1,486	3.7	19.6 D	721[e]	8.9	708	11.5	1.8 D
....	....	....	....	....	173	2.2	213	3.5	23.1 I
....	....	678	1.7	 I	....	....	56	0.9	....
47,434	100.0	39,980	100.0	15.7 D	$8,070	100.0	$6,165	100.0	23.6 D

r 1926. [d] Comparable data for electric railways not available. [e] Waterway operation and motor carriers

taining their properties, making a total of 38¢ per dollar of revenue, which compares with the 33¢ borne by the railways.

Other transport agencies have somewhat lower taxes in relation to their revenue, due principally to the fact that they do not own and maintain the right of ways or roadbed over which they operate and no taxes accrue against them on that account.

All water carriers reporting to the Interstate Commerce Commission for 1939 paid 2.7¢, and carriers on the Mississippi River System paid 2.5¢ per dollar of revenue in taxes.

Motor truck and bus companies reporting to the Interstate Commerce Commission for 1939 show a tax payment of 7.6¢ per dollar of revenue. The taxes paid by motor truck and bus lines include gasoline taxes and license fees, a large portion of which, plus other tax funds, is used for constructing, maintaining, and protecting the highways over which the carriers operate. The government owns, maintains, and protects the waterways and highways for the benefit of the users, free of charge to them and at public expense.

No portion of the taxes paid by the railways and the pipe lines is used by the government for constructing and maintaining railways and pipe lines. These taxes are used for the normal functions of government, including federal, state, county, and various municipalities. Table 9 shows a comparison of the

revenue received and taxes paid by carriers reporting to the Interstate Commerce Commission for 1939.

Regulation of Transportation Agencies

Transportation problems (largely economic) growing out of the increased activity of government, in creating and improving facilities for transportation by water, highway, and air, are becoming more complex. The growth of these various transportation agencies has been sketched briefly in this paper. On inland waterways, improved and maintained at government expense, are barges owned and operated by the government, by common-carrier operators, by contract carriers, and by private individuals or companies. On the coastal waterways, connected by canals constructed and operated by the government, are carriers whose operations are conducted under the fostering care of the government. On the public highways, built partly by gasoline taxes, are carriers operating as common carriers and under contract, and also vehicles operated solely for the use and convenience of their owners. The commercial airplane relies upon the landing facilities generally provided by public authority, and depends for its air operations upon facilities generally provided and maintained by the government.

TABLE 9.—Revenue and Taxes, 1939

No.	Description	Group A			Group B			Mississippi River system
		Railways	Pipe lines	Total	Highways	Waterways[b]	Total	
1	Revenue	$3,995,004,000	$212,466,000	$4,207,470,000	$568,481,000	$110,551,000	$679,032,000	$11,744,000
2	Taxes	355,678,000	31,936,000	387,614,000	43,298,000	2,931,000	46,229,000	289,000
3	Tax ratio[a]	0.089	0.150	0.092	0.076	0.027	0.068	0.025

[a] Taxes in dollars per dollar of revenue (item 2÷item 1). [b] All groups of waterways; includes Atlantic and Gulf coasts, Great Lakes, Mississippi River and its tributaries, and Pacific Coast carriers.

One characteristic in common is applicable to all of these agencies of transportation—their capital is mobile. If their operations prove unprofitable, they can readily be shifted to other scenes, or they can go out of business. This does not apply to the railroads, with their permanent ways and structures; nor does it apply to the pipe lines.

Until recently the problem has been further complicated by the inconsistencies of government policies. There is the related problem of adjustments in the relationships of government to a variety of interests among the several types of carriers, and the all-exclusive groups of taxpayers and of consumers whose welfare it is the duty of the government to conserve.

It has been said that the railways are the backbone of the national transportation system, but this does not deny that other forms of transportation are essential, and in fact indispensable, to the national economy and defense. Certain fundamental problems might be stated thus: (1) To define the field in which each form of transport is superior to the other for service, efficiency, and

economy; (2) to determine how services of each form operating in its proper sphere should be coordinated with the services of the other; and (3) to define the extent to which government should impose uniform regulations on all forms of transport, and special regulations, peculiar to its character, on any one form. An incidental question is whether government should undertake by regulation to confine each form of transport to its proper economic sphere or, by permitting unrestricted competition, allow each to find the field in which it can survive. Each form of transport has occupied as much of the transportation field as it could under the conditions existing today. Their services are already coordinated to a considerable degree. Government, both federal and state, has imposed regulations to a very great extent, but not equally upon the different forms of transport.

The regulation of transportation has existed in one form or another from the early time when ways were constructed by the aids of collective effort. In the United States it was not until February 4, 1887, that agreement could be reached in Congress upon a measure that purported to "regulate commerce." In effect, this first act was little more than a declaration of the intention to enter the field of railroad regulation. It created the Interstate Commerce Commission as an agency of enforcement. Since this humble beginning, the authority and powers of the Interstate Commerce Commission have been expanded gradually, until now they are very broad and affect, in varying degrees, the regulation of all forms of transportation, except airways. It was known as the Interstate Commerce Act, and it has been amended many times. Without commenting upon the changes and amendments that have been made in connection with the legislation regulating commerce, perhaps one of the most important is the Transportation Act, approved September 18, 1940. Certain provisions of this Act bear specifically on the problems affecting all transport agencies, and are of vital importance to the national economy. Quoting Title I, Section 1, under the heading, "National Transportation Policy":

> "It is hereby declared to be the national transportation policy of the Congress to provide for fair and impartial regulation of all modes of transportation subject to the provisions of this Act, so administered as to recognize and preserve the inherent advantages of each; to promote safe, adequate, economical, and efficient service and foster sound economic conditions in transportation and among the several carriers; to encourage the establishment and maintenance of reasonable charges for transportation services, without unjust discriminations, undue preferences or advantages, or unfair or destructive competitive practices; to cooperate with the several States and the duly authorized officials thereof; and to encourage fair wages and equitable working conditions;—all to the end of developing, coordinating, and preserving a national transportation system by water, highway, and rail, as well as other means, adequate to meet the needs of the commerce of the United States, of the Postal Service, and of the national defense. All of the provisions of this Act shall be administered and enforced with a view to carrying out the above declaration of policy."

This is the first time the Congress has recognized the necessity of dealing with the various transportation agencies collectively, instead of separately.

There has been no comprehensive view or proper consideration of the interrelation between the various agencies of transportation that are now competing for the same traffic throughout the United States.

For the first time in American history, subject to the provisions of the Act, the Congress is requesting a fair and impartial regulation of all agencies of transportation. The writer mentions this because airways are definitely omitted from the Act. The airways are kept separate from the others. This is a mistake because it is impossible to deal fairly with a single transportation agency without a broader view.

Another provision, under Title III, Section 301, relates to the establishment of a Board of Investigation and Research, to be composed of three persons, who shall be appointed by the President, for the purpose of investigating and reporting upon the following:

> "(1) The relative economy and fitness of carriers by railroad, motor carriers, and water carriers for transportation service, or any particular classes or descriptions thereof, with the view of determining the service for which each type of carrier is especially fitted or unfitted; the methods by which each type can and should be developed so that there may be provided a national transportation system adequate to meet the needs of the commerce of the United States, of the Postal Service, and of the national defense;
>
> "(2) The extent to which right-of-way or other transportation facilities and special services have been or are provided from public funds for the use, within the territorial limits of the continental United States, of each of the three types of carriers without adequate compensation, direct or indirect, therefor, and the extent to which such carriers have been or are aided by donations of public property, payments from public funds in excess of adequate compensation for services rendered in return therefor, or extensions of Government credit; and
>
> "(3) The extent to which taxes are imposed upon such carriers by the United States, and the several States, and by other agencies of government, including county, municipal, district, and local agencies."

The national transportation policy and the provisions of the Act to make such a study as outlined are definitely related. The policy is to be "so administered as to recognize and preserve the inherent advantages of each." What is meant by "inherent advantages"? Would not the words "relative usefulness" be more appropriate? The ability to serve all of the transportation needs of the Nation in all seasons and under all conditions should be an important consideration. The President's Board should be very helpful in determining this policy.

Pipe-line transportation is the only one that is highly specialized in the United States and, of course, no one claims that it is capable of supplanting other agencies for full community needs. It is hoped that in administering the policy, a "yardstick" will be applied that will recognize these facts.

The national policy is to promote "safe, adequate, economical, and efficient service and foster sound economic conditions in transportation and among the several carriers." The terms "safe, adequate, economical, and efficient"

require no comment. However, the meaning of "sound economic conditions" may be vague when applied to the several transportation agencies. Is it the intent of the Congress that all agencies of commercial transportation should be self-supporting, or does "sound economic conditions" imply a continued government aid to some forms of transportation?

It would appear that, under Title III, Section 301, of the Act, the President's Board will be expected to weigh all of the factors affecting the economies of the railway, highway, and waterway carriers; it will consider all costs of transportation—those borne by the shippers and those borne by the taxpayers.

The national transportation policy is "to encourage the establishment and maintenance of reasonable charges for transportation services, without unjust discriminations, undue preferences or advantages, or unfair or destructive competitive practices." If fair and impartial application is made of these principles, many of the important factors that are effecting chaos will be eliminated.

The declaration, "all to the end of developing, coordinating, and preserving a national transportation system by water, highway, and rail, as well as other means, adequate to meet the needs of the commerce of the United States, of the Postal Service, and of the national defense" is of interest. It is the opinion of many that there are too many transportation facilities. The national transportation burden is now at its height, for that reason. The three agencies mentioned are more than adequate to meet the needs of commerce, and the government continues to develop more facilities by water, highway, and air. What about national defense? The railway is indispensable, as has been testified by competent Army officers. If additional facilities are necessary for this purpose, a much greater capacity can be developed for the least expenditure, in rail facilities, as compared with waterway and highway expenditures. The Army engineers in charge of waterway investments, from the beginning, have given little consideration, if any, to existing facilities, when contemplating the extension of waterway facilities. The economic justification rests upon diverting traffic from existing facilities to the waterways. Coordination cannot come out of such a policy. No doubt the President's Board will weigh very carefully all of the many controversial factors affecting this problem.

Transportation Costs

This paper will not be complete without a brief discussion of transportation costs as may be considered by the President's Board of Investigation and Research. Among other things, it will be authorized to ascertain, if possible, "The relative economy and fitness of carriers by railroad, motor carriers, and water carriers for transportation service, or any particular classes or descriptions thereof, with the view of determining the service for which each type of carrier is especially fitted or unfitted." One of the most important factors to be considered by the Board will be the actual cost of producing transportation by these types of carriers. This will be most difficult, if not impossible, to accomplish. Railway cost engineers and the Statistical Division of the Interstate Commerce

Commission have had this question before them for many years, as it applies to the railroads, and in recent years to the highways, and up to now (1941) the results have not been comforting. There may be more than one method of approach to this problem. The one that has been so complex and difficult is to determine what it costs to transport persons and property (classified as to commodities) by the various transport agencies without regard to the amounts paid by the users. This method meets with very serious handicaps, because of the impossibility of determining the costs as applied to individual persons and various kinds of property. This would be particularly true in connection with railway, waterway, highway, and airway transportation. Oil and gasoline constitute the major part of property transported in pipe lines in a fluid form, which, of course, simplifies the problem of cost finding. With the railways, waterways, highways, and airways, all of which transport many classes of commodities, the problem has not been solved. It naturally follows that, if this method of approach is used, it will be necessary to determine costs applicable to passenger, mail, and express; costs of l. c. l. (less than carload-lots) packages of all sizes and weights, and freight of all kinds moving in carloads, for long hauls and for short hauls (all mixed up in long trains and short trains, in large or small motor vehicles, steamboats and barge tows, and air transports) and for varying operating conditions as may obtain throughout the United States.

All transportation agencies are competing for business by means of a pricing system and service offering that have resulted in dividing the potential traffic among the various agencies. Records are available of the revenues and expenses of all common carriers reporting to public authority. The differential between the total revenues and total expenses of the carriers is so narrow that the price paid by the shipper might properly be considered as the cost of transportation.

The amount of government aid, particularly in connection with the highway carriers, is not easy to determine. A number of studies have already been made for the purpose of finding out if the commercial users of the highways are paying their full share. The highway carriers claim that the various tax levies plus gasoline taxes more than compensate for the use of the highways. The railroads do not agree. When the total of public aid contributions is added to the total amount of the carriers' expenses, or the total amount paid the carriers by the shippers, it should not be difficult to determine the relative total economy of the various types of transport agencies.

With accurate data as to the amount of government aid affecting transportation, it would then become a matter of public policy as to whether or not such aid should be continued or withdrawn. If all forms of government aid were withdrawn from commercial transportation agencies, economic laws would soon point out which form of transportation was the most economical and satisfactory to the users. The elimination of government aids, except for experimental, promotional, and defense purposes, would solve the problem now so vitally affecting national transportation. Therefore, it seems logical that the first thing the President's Board should do is to develop, very carefully, the

amounts of government aid now reflected in the cost of commercial transportation in all of its forms. Much has been accomplished already in this respect.

Joseph B. Eastman, when federal coordinator, evidently had in mind the importance of the question of subsidy, or government aid, to the various transportation agencies when he undertook to prepare the four large volumes of "Public Aids to Transportation," which were begun, through his Research Section, in 1933–1936, and later completed and issued by the chairman of the Interstate Commerce Commission in 1940. This report contains a great wealth of valuable information, together with comments and conclusions.

In October, 1940, J. J. Pelley, President of the Association of American Railroads, released a reply entitled "What Is Public Aid to Transportation?" (produced and distributed by the Association of American Railroads), which is an analysis of the data contained in Mr. Eastman's report, and points to inconsistencies that render the conclusions incomparable, confusing, and misleading.

Here are the results of two studies that have a direct bearing on the future work of the President's Board of Investigation and Research, and it will be well for all who are interested to study in great detail the information contained in both of these reports.

Acknowledgments

Credit is due the Association of American Railroads; Brookings Institute ("The American Transportation Problem"); Federal Works Agency, Public Roads Administration; Interstate Commerce Commission; Office of Chief of Engineers, U.S. Army; U. S. Department of Commerce, Civil Aeronautics Administration; and others for information contained in this paper. Fig. 1 was supplied by the Southern Pacific Railway Company, and Fig. 4 by the Pennsylvania Railroad Company.

AMERICAN SOCIETY OF CIVIL ENGINEERS

Founded November 5, 1852

PAPERS

LATERAL STABILITY OF UNSYMMETRICAL I-BEAMS AND TRUSSES IN BENDING

By George Winter,[1] Esq.

Synopsis

An analysis is given herein of the lateral stability of unsymmetrical I-beams, both free and horizontally restrained along the bottom edge, and of beams of rectangular cross section horizontally restrained along the bottom edge. The cases of horizontally restrained beams apply to sheet stiffeners in bending. It is explained that the torsional rigidity of trusses and of thin-wall I-beams is smaller than would be expected from the use of the usual formulas for torsional stiffness. This is due to the fact that horizontal bending of such elements is accompanied by distortion of the cross section. For this reason, it is suggested that satisfactory, although slightly conservative, estimates of the critical loads of such beams may be obtained by neglecting the torsional rigidity entirely.

The Problem

The solutions of the problem of lateral instability of beams known in the literature do not cover the entire range of cases arising in practice. A few of these are:

(1) The general case of an I-beam non-symmetrical about its horizontal axis;

(2) The case of beams of various cross sections horizontally restrained along the tension edge (this situation is realized for sheets stiffened by projecting ribs and subject to bending in such a way that the sheet is in tension); and

(3) The lateral stability of trusses and of I-beams made of thin sheet metal.

These problems are analyzed herein for the case of pure bending and for the case of a simple beam with a concentrated load at the center of the span, applied at any arbitrary level.

Note.—Written comments are invited for immediate publication; to insure publication the last discussion should be submitted by April, 1942.

[1] Research Associate, School of Civ. Eng., Cornell Univ., Ithaca, N. Y.

Notation.—The letter symbols in this paper are defined where they first appear, and are assembled for reference in Appendix I.

Method

The work-energy method is used throughout this investigation. The horizontal displacement of the beam due to lateral instability is resolved into two components (Fig. 1): A horizontal bending displacement y of the flexural center, and a rotation ϕ about a horizontal axis through the flexural center.

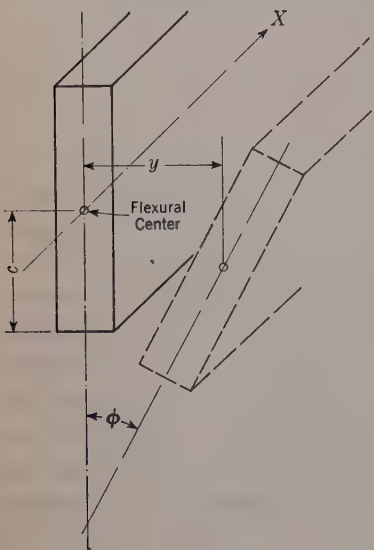


Fig. 1

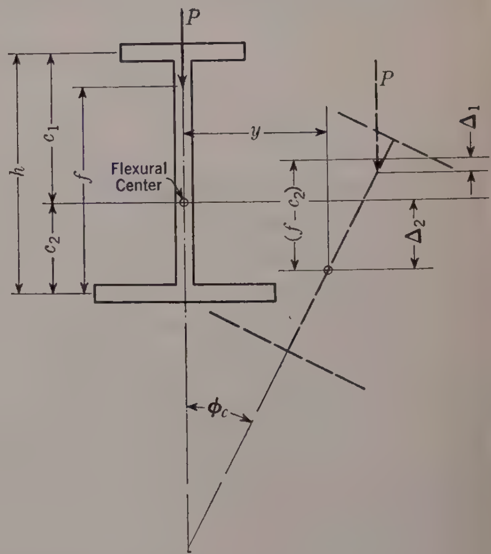


Fig. 2

The angle ϕ is an unknown function of the longitudinal ordinate x (origin at one of the supports). The beam is assumed to be supported in such a way that rotation of the end sections about the longitudinal axis of the beam is prevented. Consequently, the condition that must be satisfied by ϕ at the ends is: $\phi = 0$ at $x = 0$ and $x = L$.

Empirically it is known that, after loss of stability, the horizontal deformation of the beam has the form of a single half wave. This general character is represented and the boundary conditions are satisfied by taking

$$\phi = a \sin \frac{\pi x}{L} \qquad (1)$$

in which a = a coefficient modifying the total angular deformation ϕ (see Eq. 1); and L = span length.

The assumption of this approximate form of $\phi(x)$ involves a certain error, the magnitude of which could be estimated by expanding $\phi(x)$ in a Fourier series with unknown coefficients instead of taking only the first term of such a series. Without making use of this elaborate procedure it will be shown by comparison that the error involved is negligible for all practical purposes.

Values of Integrals Frequently Arising in the Analysis

To simplify the mathematical presentation, the solutions of definite integrals which will frequently appear in the following pages are given as follows:

$$\left.\begin{aligned} \int_0^L \phi^2\, dx &= \frac{a^2 L}{2} \\ \int_0^L \left(\frac{d\phi}{dx}\right)^2 dx &= a^2 \frac{\pi^2}{2L} \\ \int_0^L \left(\frac{d^2\phi}{dx^2}\right)^2 dx &= a^2 \frac{\pi^4}{2L^3} \\ \int_0^{L/2} \phi \frac{d^2\phi}{dx^2} x\, dx &= -\frac{a^2}{4}\left(1 + \frac{\pi^2}{4}\right) \\ \int_0^{L/2} \phi^2 x^2\, dx &= \frac{a^2 L^3}{8\pi^2}\left(1 + \frac{\pi^2}{6}\right) \end{aligned}\right\} \qquad (2)$$

I-Beams Asymmetrical About the Horizontal Axis

(*a*) *Pure Bending.*—For such beams the location of the flexural center is given by:[2]

$$c_1 = h \frac{I_2}{I} \qquad (3a)$$

and

$$c_2 = h \frac{I_1}{I} \qquad (3b)$$

in which (see Fig. 2) c_1 and c_2 are the distances to the flexural center from the centers of the compression and tension flanges of the beam, respectively; $h = c_1 + c_2 =$ the total effective depth; and I, I_1, and I_2, respectively, are the inertia moments of the total beam, the compression flange, and the tension flange with respect to the vertical axis of the web. In computing I the moment of the web about its vertical axis is neglected.

The strain energy of the beam due to lateral bending and rotation about its flexural center consists of three parts (Fig. 2): (*a*) The energy of bending of the compression flange corresponding to a horizontal deflection $y + c_1 \phi$; (*b*) the energy of bending of the tension flange corresponding to a horizontal deflection $y - c_2 \phi$; and (*c*) the torsional energy corresponding to a rotation of the entire beam through the angle ϕ. Consequently, the strain energy is

$$W_\epsilon = \tfrac{1}{2}\left\{ E I_1 \int_0^L \left[\frac{d^2(y + c_1 \phi)}{dx^2}\right]^2 dx \right.$$
$$\left. + E I_2 \int_0^L \left[\frac{d^2(y - c_2 \phi)}{dx^2}\right]^2 dx + G K \int_0^L \left(\frac{d\phi}{dx}\right)^2 dx \right\} \qquad (4)$$

[2] "Formulas for Stress and Strain," by R. J. Roark, McGraw-Hill Book Co., Inc., New York, N. Y., 1938, p. 118.

or, by substitution of c_1 and c_2 from Eq. 3,

$$W_\epsilon = \frac{E}{2}\left[I\int_0^L\left(\frac{d^2y}{dx^2}\right)^2 dx + \frac{I_1 I_2}{I}h^2\int_0^L\left(\frac{d^2\phi}{dx^2}\right)^2 dx + C\int_0^L\left(\frac{d\phi}{dx}\right)^2 dx\right] \quad (5)$$

In Eq. 4, K = the torsional constant for the given cross section, G = the shear modulus—

$$G = \frac{E}{2\,(1+\mu)} \quad (6)$$

and μ = Poisson's ratio. In Eq. 5, C, the reduced torsional constant, equals

$$C = \frac{K}{2\,(1+\mu)} \quad (7)$$

If M is the vertical bending moment, its horizontal component acting on the beam in the displaced position is $M_y = M\,\phi$ and

$$\frac{d^2y}{dx^2} = -\frac{M\,\phi}{E\,I} \quad (8)$$

Substituting this expression for $\frac{d^2y}{dx^2}$ and also the appropriate values for the integrals, one arrives at

$$W_\epsilon = E\frac{a^2}{2}\left(\frac{M^2}{E^2 I}\frac{L}{2} + \frac{I_1 I_2}{I}h^2\frac{\pi^4}{2\,L^3} + C\frac{\pi^2}{2\,L}\right) \quad (9)$$

The only external forces acting on the beam are the moments M applied at the ends (Fig. 3). If β designates the vertical angle of rotation of each of the

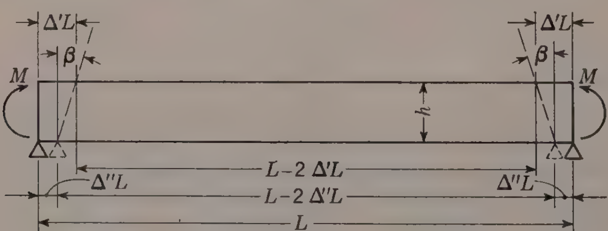


Fig. 3

end sections due to the horizontal deformation of the beam, the work of the external forces is

$$W = 2\,\beta\,M \quad (10)$$

The magnitude of β is obtained by determining half of the difference in length of the chords of the horizontal curves formed by the axes of the top and bottom flanges in the laterally bent state ($\Delta' L - \Delta'' L$ in Fig. 3). Thus,

$$\beta = \frac{1}{2\,h}\left\{\int_0^{L/2}\left[\frac{d(y + c_1\,\phi)}{dx}\right]^2 dx - \int_0^{L/2}\left[\frac{d(y - c_2\,\phi)}{dx}\right]^2 dx\right\}; \text{ or}$$

$$\beta = \tfrac{1}{4}\left[2\int_0^L \frac{dy}{dx}\frac{d\phi}{dx}\,dx + h\,\frac{I_2 - I_1}{I}\int_0^L\left(\frac{d\phi}{dx}\right)^2 dx\right\} \quad (11)$$

From Eqs. 1 and 8, however,

$$\frac{dy}{dx} = \frac{a\,M}{E\,I}\left(\frac{L}{\pi}\cos\frac{\pi\,x}{L}\right) \qquad (12)$$

Since $W = 2\,\beta\,M$ (Eq. 10) one obtains, by substitution of the appropriate values for the integrals,

$$W = a^2\,\frac{M}{2\,I}\left[h\,(I_2 - I_1)\,\frac{\pi^2}{2\,L} + \frac{M\,L}{E}\right] \qquad (13)$$

The critical moment is now determined from the work-energy condition, $W_\epsilon = W$, which results in

$$M_{cr} = E\,h\,\frac{\pi^2}{2\,L^2}\left[(I_1 - I_2) + \sqrt{I^2 + 4\,C\,I\,\frac{L^2}{\pi^2\,h^2}}\right] \qquad (14a)$$

For the case of a symmetrical I-beam with $I_1 = I_2 = I_0$, Eq. 14*a* reduces to

$$M_{cr} = E\,\frac{\pi^2}{L^2}\sqrt{I^2{}_0\,h^2 + 2\,C\,I_0\,\frac{L^2}{\pi^2}} \qquad (14b)$$

The exact solution for this latter special case has been developed by S. Timoshenko. If Professor Timoshenko's value of $\frac{a^2}{L^2}$ is substituted[3] in his comparable equation[4] for M_{cr}, the result is identical with Eq. 14*b*. In other words, for the case of pure bending, Eq. 1 happens to be the exact expression for $\phi(x)$.

(*b*) *Center Load*.—Eq. 5 for the strain energy remains the same as in the previous case. Between $x = 0$ and $x = \frac{L}{2}$,

$$\frac{d^2y}{dx^2} = -\frac{M\,\phi}{E\,I} = -\frac{P\,x\,\phi}{2\,E\,I} \qquad (15)$$

in which P = a vertical concentrated load applied at an arbitrary distance f from the bottom flange. Upon substitution of this expression the first integral in Eq. 5 becomes

$$\int_0^L \left(\frac{d^2y}{dx^2}\right)^2 dx = \frac{P^2}{2\,E^2\,I^2}\int_0^{L/2} \phi^2\,x^2\,dx \qquad (16)$$

If the corresponding values of the integrals are introduced, the strain energy in this case is

$$W_\epsilon = a^2\left[\frac{P^2}{E\,I}\,\frac{L^3}{32\,\pi^2}\left(1 + \frac{\pi^2}{6}\right) + h^2\,\frac{I_1\,I_2}{I}\,\frac{\pi^4}{4\,L^3}\,E + \frac{C\,E}{4}\,\frac{\pi^2}{L}\right] \qquad (17)$$

The vertical displacement of the force P, applied at an arbitrary distance f from the bottom flange, may be resolved into two components, Δ_1 and Δ_2.

---

[3] "Theory of Elastic Stability," by S. Timoshenko, McGraw-Hill Book Co., Inc., New York, N. Y., Eq. 157, p. 258.

[4] *Ibid.*, Eq. 159, p. 258.

The component Δ_1 (Fig. 2) represents the relative displacement of the point of load application with respect to the flexural center. It is easily seen geometrically that the amount of this displacement is

$$\Delta_1 = (f - c_2)\,(1 - \cos \phi_c) \ldots\ldots\ldots\ldots (18)$$

in which ϕ_c is the angle ϕ at the center of the span. Developing $\cos \phi$ in a power series in ϕ and, for a small value of ϕ, retaining only the first two terms:

$$\Delta_1 = a^2 \frac{f - c_2}{2} \ldots\ldots\ldots\ldots (19)$$

The second component Δ_2 represents the relative displacement of the flexural center with respect to the supports caused by the horizontal curvature of the longitudinal axis of the beams through the flexural center. Thus

$$\Delta_2 = -\int_0^{L/2} \frac{d^2y}{dx^2}\,\phi\,x\,dx = \frac{P}{2\,E\,I}\int_0^{L/2} \phi^2\,x^2\,dx \ldots\ldots\ldots\ldots (20)$$

Consequently, the work of the force P is $W = P\,(\Delta_1 + \Delta_2)$ or,

$$W = \left[\frac{P^2}{E\,I}\,\frac{L^3}{16\,\pi^2}\left(1 + \frac{\pi^2}{6}\right) + \frac{P}{2\,I}\,(f\,I - h\,I_1)\right] a^2 \ldots\ldots\ldots\ldots (21)$$

The critical load is again obtained by making $W_\epsilon = W$, which yields

$$P_{cr} = \frac{48\,\pi^2\,E}{L^3\,(6 + \pi^2)}\left[(h\,I_1 - f\,I) + \right.$$

$$\left. + \sqrt{(h\,I_1 - f\,I)^2 + \frac{L^2\,(6 + \pi^2)}{48}\left(C\,I + \frac{h^2\,I_1\,I_2\,\pi^2}{L^2}\right)}\,\right] \ldots\ldots (22a)$$

For the special case of the symmetrical I-beam ($I_1 = I_2 = I_0$) with P applied at the centroid $\left(f = \frac{h}{2}\right)$ this expression simplifies to

$$P_{cr} = \frac{12\,\pi^2}{L^2}\,E\,\sqrt{\frac{I_0}{3\,(6 + \pi^2)}\left(2\,C + \frac{h^2\,\pi^2}{L^2}\,I_0\right)} \ldots\ldots\ldots\ldots (22b)$$

The exact solution for this special case in explicit form is not known to the writer. Using the energy method, Professor Timoshenko derived an integral equation[5] for this case. In solving it he uses only the first term of a Fourier expansion[6] which is identical with Eq. 1. It can be shown easily that, in this case, Professor Timoshenko's solution is identical with Eq. 22*b*. According to Professor Timoshenko, the error involved in taking this approximate expression for $\phi(x)$ amounts to only 0.5% in this special case. Without further detailed investigation it is reasonable to state that, in the general case of an unsymmetrical I-beam, loaded at any arbitrary level f, the error involved

---

[5] "Theory of Elastic Stability," by S. Timoshenko, McGraw-Hill Book Co., Inc., New York, N. Y., Eq. e, p. 266.

[6] *Ibid.*, Eq. f, p. 266.

is of the same order of magnitude, and is therefore negligible for all practical purposes.

Rectangular Beams Horizontally Restrained Along the Tension Edge

Imagine a simply supported rectangular beam restrained along its bottom (tension) edge in such a way that horizontal displacements are prevented, whereas vertical deflections remain entirely unaffected. This case is very nearly realized if, for some practical reason, horizontal bracing is applied only along the bottom edge. It is granted that such an arrangement in practice will be used only in exceptional cases. However, this problem is not only of general interest, but it leads to the analysis of the more important question of stability of sheet stiffeners of I-section and T-section, which will be treated subsequently.

(*a*) *Pure Bending*.—The horizontal displacement of the beam (Fig. 4) is again resolved into the horizontal deflection y of its flexural center and into a rotation about the same point. The general expression for the strain energy is then

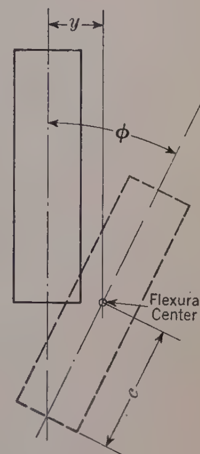


Fig. 4

$$W_\epsilon = \frac{E}{2}\left[I\int_0^L \left(\frac{d^2y}{dx^2}\right)^2 dx + C\int_0^L \left(\frac{d\phi}{dx}\right)^2 dx\right] \quad (23)$$

As seen from Fig. 4 for small angles ϕ

$$y = c\,\phi \quad (24)$$

Substituting Eq. 24 in Eq. 23 yields

$$W_\epsilon = \frac{E}{2}\left[I\,c^2\int_0^L \left(\frac{d^2\phi}{dx^2}\right)^2 dx + C\int_0^L \left(\frac{d\phi}{dx}\right)^2 dx\right] \quad (25)$$

On the basis of the same considerations as applied before to I-beams, the angle of rotation β of the end section is

$$\beta = \frac{1}{2\,c}\int_0^{L/2} \left(\frac{dy}{dx}\right)^2 dx = \frac{c}{2}\int_0^{L/2} \left(\frac{d\phi}{dx}\right)^2 dx \quad (26)$$

and the work of the external moments M becomes

$$W = 2\,\beta\,M = \frac{M\,c}{2}\int_0^L \left(\frac{d\phi}{dx}\right)^2 dx \quad (27)$$

It should be noted that in this case the moments M are not the only external forces acting on the beam. In fact, along the bottom edge horizontal forces are exerted by the restraint, which are of such magnitude that they prevent horizontal displacement of this edge. However, since the points of application of these forces are not subject to any displacement in the direction of the forces,

they do no work. For this reason, Eq. 27 gives the total amount of work of the external forces.

It is well established, empirically and analytically, that laterally unrestrained beams buckle in a single half wave. For this reason it was legitimate to use Eq. 1 for an approximate representation of ϕ for free beams. However, in the case of beams restrained laterally along the bottom edge, it is possible that buckling may occur in more than one half wave. Buckling of such multiwave character is known to occur in thin plates, struts on elastic supports, etc.

To investigate this possibility, one must assume the most general expression for $\phi(x)$ that will satisfy the end conditions. Such a general expression is the Fourier series

$$\phi = \sum_{n=1,2,\cdots}^{\infty} a_n \sin \frac{n \pi x}{L} \qquad (28)$$

in which a_n represents undetermined coefficients. Then

$$\int_0^L \left(\frac{d\phi}{dx} \right)^2 dx = \frac{\pi^2}{2L} \sum_{n=1}^{\infty} n^2 a^2_n \qquad (29)$$

and

$$\int_0^L \left(\frac{d^2\phi}{dx^2} \right)^2 dx = \frac{\pi^4}{2L^3} \sum_{n=1}^{\infty} n^4 a^2_n \qquad (30)$$

Substituting these values in Eqs. 25 and 27 and equating $W_\epsilon = W$, one arrives at

$$M_{cr} = E \frac{I c \frac{\pi^2}{L^2} \sum_{n=1}^{\infty} n^4 a^2_n + \frac{C}{c} \sum_{n=1}^{\infty} n^2 a^2_n}{\sum_{n=1}^{\infty} n^2 a^2_n} \qquad (31)$$

All possible values of a^2_n are fundamentally positive quantities. Consequently, the smallest value of M_{cr} corresponds to $n = 1$. It is thus shown that a laterally restrained beam buckles in a single half wave; in addition, in the particular case of pure bending, Eq. 1 is seen to be the exact expression for the shape of the beam.

Therefore, with $n = 1$,

$$M_{cr} = E \left(c \frac{\pi^2}{L^2} I + \frac{C}{c} \right) \qquad (32a)$$

Since, for a rectangular beam, $c = \frac{h}{2}$,

$$M_{cr} = E \left(h \frac{\pi^2}{2L^2} I + \frac{2c}{h} \right) \qquad (32b)$$

(*b*) *Center Load.*—The expression for the strain energy is the same as in the previous case (Eq. 25).

The vertical displacement of the load P is obtained by the same reasoning as in the case of I-beams. Thus (see Eq. 19):

$$\Delta_1 = a^2 \frac{f - c}{2} \qquad (19)$$

$$\Delta_2 = -c \int_0^{L/2} \phi \frac{d^2\phi}{dx^2} x\, dx \qquad (33)$$

and

$$W = P \left(a^2 \frac{f - c}{2} - c \int_0^{L/2} \phi \frac{d^2\phi}{dx^2} x\, dx \right) \qquad (34)$$

From $W_\epsilon = W$, the following expression for the critical load is obtained:

$$P_{cr} = E \frac{\pi^2}{L} \frac{I\, c^2 \dfrac{\pi^2}{L^2} + C}{2 f + c \left(\dfrac{\pi^2}{4} - 1 \right)} \qquad (35a)$$

or, since for a rectangle, $c = \dfrac{h}{2}$,

$$P_{cr} = E \frac{\pi^2}{L} \frac{\left(\dfrac{\pi\, h}{2\, L} \right)^2 I + C}{2 f + 0.5\, L \left(\dfrac{\pi^2}{4} - 1 \right)} \qquad (35b)$$

It may be noted that both Eq. 32*a* and Eq. 35*a* also apply to cross sections other than rectangles, provided that the nature of the section is such that its total resistance to torsion is proportional to K only. This is generally true for cross sections that do not have transversely projecting parts (flanges, etc.).

I-Beams Horizontally Restrained Along Tension Flange

This case is realized for sheets or plates stiffened by projecting ribs of I-beam or T-beam shape and subject to bending of such nature that the sheet or plate is in tension. The sheet is then to be regarded as an integral part of the stiffener—that is, it is to be included in the determination of the moment of inertia I_2 of the tension flange. Whether or not it is also to be included in the determination of the torsional constant K depends upon the type of joint between stiffener and sheet. For joints acting rather in a hinge-like manner (that is, allowing rotation of the stiffeners with respect to the sheet), K should be taken for the stiffener only. The same is true for rather thin sheets (as compared with the wall thickness of the stiffener), since in this case the sheet is likely to bend slightly out of its plane in the immediate vicinity of the joint rather than to twist monolithically with the stiffener.

(*a*) *Pure Bending.*—It is seen from Fig. 5 that, in this case, for small angles ϕ,

$$y = c_2 \phi \qquad (36)$$

Substituting Eq. 36 in Eq. 5 and remembering that

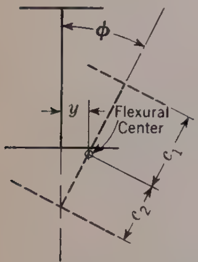


FIG. 5

$$c_2 = h \frac{I_1}{I} \qquad (37)$$

the strain energy (after introducing the corresponding values for the integrals) becomes

$$W_\epsilon = E \frac{\pi^2 h^2}{4 L} \left(I_1 \frac{\pi^2}{L^2} + \frac{C}{h^2} \right) a^2 \qquad (38)$$

On the other hand, substituting Eq. 36 in Eq. 11 for β, one arrives at

$$W = \frac{h M}{4} \frac{\pi^2}{L} a^2 \qquad (39)$$

Again, making $W_\epsilon = W$, it follows that the critical moment

$$M_{cr} = E h \left(I_1 \frac{\pi^2}{L^2} + \frac{C}{h^2} \right) \qquad (40)$$

Eq. 40 is seen to be independent of I_2, because the tension flange is not subject to any horizontal displacement and the position of the external forces (that is, M) is in no way connected with the position of the flexural center. For this reason Eq. 40 applies to symmetrical as well as to unsymmetrical **I**-beams.

(*b*) *Center Load.*—Eq. 38 for the strain energy remains the same as in the previous case. The vertical displacement of the load, with $y = c_2 \phi$, becomes

$$\Delta = \Delta_1 + \Delta_2 = a^2 \frac{f - c_2}{2} - c_2 \int_0^{L/2} \phi \frac{d^2\phi}{dx^2} x \, dx \qquad (41)$$

and the work of the external force P

$$W = \frac{P}{2 I} \left[f I + \frac{h I_1}{2} \left(\frac{\pi^2}{4} - 1 \right) \right] a^2 \qquad (42)$$

From $W_\epsilon = W$

$$P_{cr} = E I \frac{\pi^2 h^2}{2 L} \frac{I_1 \dfrac{\pi^2}{L^2} + \dfrac{C}{h^2}}{f I + \dfrac{h I_1}{2} \left(\dfrac{\pi^2}{4} - 1 \right)} \qquad (43a)$$

and for the special case of $I_1 = I_2 = I_0, f = \frac{h}{2}$,

$$P_{cr} = 2 E h \frac{\pi^2}{L} \frac{I_0 \dfrac{\pi^2}{L^2} + \dfrac{C}{h^2}}{\dfrac{\pi^2}{4} + 1} \qquad (43b)$$

Trusses and I-Beams with Thin Webs

These elements require special consideration for the following reason: In general the critical load depends upon the horizontal flexural rigidity and upon the torsional rigidity of the beam. For ordinary beams with thick webs the torsional constant may be determined from well-known formulas.[7] These constants have been derived on the assumption that the shape of the cross section does not change during torsion, or, in other words, that all component parts of the cross section are rotated through the same angle. This is not the case for latticed trusses, however (see Fig. 6(*a*)), and for beams with thin webs (see Fig. 6(*b*)). In such elements the lateral rigidity of the web, in many cases, is so slight that it bends out of its plane when the beam becomes unstable. For this reason the angle of rotation α_1 of the chords or flanges is seen to be less than the angle of rotation α_2 of the line AB in Fig. 6(*c*). Consequently the actual torsional rigidity of such elements will be less than that obtained from a formal application of the usual formulas for the torsional constant K. The determination of the actual torsional resistance of such members would require highly involved computations in each particular case and thus is beyond the scope of this paper.

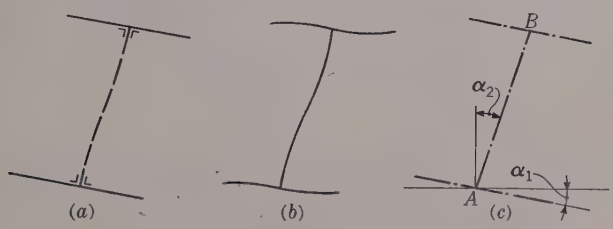


Fig. 6

However, for beams of this type, if K' is computed on the basis of the usual formulas (that is, without considering the distortion of the cross section) it will be found that the contribution to the critical load of the lateral flexural rigidity $E\,I$ is usually far in excess of the contribution of the torsional rigidity $G\,K$ to the critical load. Since the actual torsional constant K for beams of this type usually is considerably less than K' derived by formula, it seems advisable to determine the critical load of such beams by disregarding K entirely (that is, by assuming $K = 0$). The value thus obtained will be on the conservative side. On the other hand, if the effect of distorting the cross section were disregarded (that is, if K', determined by formula, were used instead of the unknown actual K), the computed critical loads would be decidedly in excess of the actual loads. It is believed, therefore, that the first of these procedures (that is, assuming $K = 0$ for beams of this type) is by far preferable in current design practice.

[7] "Formulas for Stress and Strain," by R. J. Roark, McGraw-Hill Book Co., Inc., New York, N. Y., 1938, p. 154.

Conclusion

In order to facilitate the use of the results of this investigation for beams with very small torsional constants, the formulas presented in this paper are assembled for reference in Appendix II.

APPENDIX I

Notation

The following letter symbols, adopted for use in this paper, conform essentially to Standard Letter Symbols for Structural Analysis, prepared by a Committee of the American Standards Association with Society representation and approved by the Association in 1941.[8]

a = amplitude of the angular deformation: a_n = an undetermined coefficient;
C = reduced torsional constant for a given section (see Eq. 7);
c = vertical distance of the flexural center of a rectangular beam from the bottom (tension edge) (see Fig. 1):
  c_1 = distance from the center of the compression flange;
  c_2 = distance from the center of the tension flange (Fig. 2);
E = Young's modulus;
f = vertical distance from the bottom flange to the point of application of P;
G = shear modulus (Eq. 6);
h = effective height of section, between centers of flanges $(c_1 + c_2)$;
I = total moment of inertia $(I_1 + I_2)$ of a cross section with respect to the vertical axis of symmetry, particularly for I-beams (the moment of inertia of the web about its vertical axis is ignored):
  I_1 = inertia moment of the compression flange about the same axis;
  I_2 = inertia moment of the tension flange about the same axis;
K = torsional constant for a given cross section: K' = a computed constant as compared with (K) an actual constant;
L = span length, between supports:
  $\Delta' L$ = longitudinal displacement of the top (tension) flange, at the support;
  $\Delta'' L$ = longitudinal displacement of the bottom (compression) flange, at the support;
M = vertical bending moment:
  M_{cr} = critical moment;
  M_y = lateral component of M;
P = a concentrated external load (Eq. 5);
W = work of external forces: W_ϵ = strain energy;

---

[8] Publication pending.

x = a longitudinal ordinate (see Fig. 1);
y = deflection, horizontal bending displacement (see Fig. 1);
α = angle of rotation:
 α_1 = rotation of the chords or flanges of an I-beam;
 α_2 = rotation of the web (Fig. 6);
β = vertical angle of rotation of the end section (Fig. 3);
Δ = total linear deformation (see also ΔL):
 Δ_1 = relative displacement of the point of application of the load P, with respect to the flexural center;
 Δ_2 = relative displacement of the flexural center with respect to the support, caused by the horizontal curvature of the longitudinal axis of the beams through the flexural center;
μ = Poisson's ratio;
ϕ = total angular deformation (Fig. 1): ϕ_c = angle ϕ at the center of the span.

APPENDIX II

Design Equations

The design equations discussed in some detail in the paper are Eqs. 14, 22, 32, 35, 40, and 43. For beams with very small torsional rigidity, in particular for open web trusses and thin wall beams, assuming in this case $K = 0$, one obtains the following simplified expressions—

Unsymmetrical I-beams, pure bending, unrestrained:

$$M_{cr} = E\,h\,I_1 \frac{\pi^2}{L^2} \qquad (44a)$$

Symmetrical I-beam, pure bending, unrestrained:

$$M_{cr} = E\,h\,I_0 \frac{\pi^2}{L^2} \qquad (44b)$$

Unsymmetrical I-beam, center load, unrestrained:

$$P_{cr} = \frac{48\,\pi^2\,E}{L^3\,(6+\pi^2)}\left[(h\,I_1 - f\,I) + \sqrt{(h\,I_1 - f\,I)^2 + \frac{6+\pi^2}{48}\,h^2\,\pi^2\,I_1\,I_2}\right] \qquad (45a)$$

Symmetrical I-beam, center load, $f = \frac{h}{2}$, unrestrained:

$$P_{cr} = E\,h\,I_0 \frac{\pi^3}{L^3}\sqrt{\frac{48}{6+\pi^2}} \qquad (45b)$$

Unsymmetrical I-beam, pure bending, restrained:

$$M_{cr} = E\,h\,I_1 \frac{\pi^2}{L^2} \qquad (46a)$$

Symmetrical **I**-beam, pure bending, restrained:

$$M_{cr} = E\,h\,I_0 \frac{\pi^2}{L^2} \quad \text{.....................} (46b)$$

Unsymmetrical **I**-beam, center load, restrained:

$$P_{cr} = E\,I \frac{\pi^3}{2\,L^3}\,h^2 \frac{I_1\,\pi}{f\,I + \frac{h\,I_1}{2}\left(\frac{\pi^2}{4} - 1\right)} \quad \text{.............} (47a)$$

Symmetrical **I**-beam, center load, $f = \frac{h}{2}$, restrained:

$$P_{cr} = E\,I \frac{\pi^3}{L^3}\,h \frac{2\,\pi}{\frac{\pi^2}{4} + 1} \quad \text{...................} (47b)$$

The formal requirements for the use of these formulas are: For Eqs. 44 and 45*b*, $C \ll \frac{I}{4}\left(\frac{\pi\,h}{L}\right)^2$; for Eq. 45*a*, $C \ll \frac{I_1\,I_2}{I}\left(\frac{\pi\,h}{L}\right)^2$; and for Eqs. 46 and 47, $C \ll I_1 \left(\frac{\pi\,h}{L}\right)^2$ ($\ll$ denotes "much less than").

As stated previously, in beams of this type, the actual value of K is generally unknown and it is only evident that $K < K'$. For this reason these formal requirements should be used with critical care as guides rather than as rigorously applicable criteria.

AMERICAN SOCIETY OF CIVIL ENGINEERS

Founded November 5, 1852

DISCUSSIONS

FUNDAMENTAL ASPECTS OF THE DEPRECIATION PROBLEM A SYMPOSIUM

Discussion

By Messrs. William G. Atwood, George E. Goldthwaite, Nathan B. Jacobs, H. J. Flagg, Nelson Lee Smith, and C. Beverley Benson

William G. Atwood,[17] M. Am. Soc. C. E.[17a]—The three papers presented in this Symposium show, as is realized by all students of the problem, that a precise definition for the term "depreciation" is the first requisite for a study of the subject. Mr. Walls makes this clear in his paper and suggests an approach. There is a great mass of literature on the subject, most of which has been written by authors with a limited class of problems in mind. After studying much of this material the reader cannot "see the forest for the trees."

Depreciation is a physical and economic fact which, in most cases, must finally be measured in dollars. In order to determine the number of dollars it is generally first necessary to find the percentage of value that remains. The dollars are merely markers. The use of the term "depreciation" in accounting is one of the causes of the difficulty in defining the term correctly. If instead of "depreciation" the account were called "amortized cost," as suggested by Professor Grant, one of the difficulties would be removed.

"Depreciation" charges in accounting are generally based on the "straight-line" or "age-life" theory. The writer agrees with Mr. Walls that this method does not show the facts correctly. Perhaps, among the reasons for its popularity is that it is simple and arithmetical and that it can be readily understood by the courts and commissions. It is probably necessary to use this method for accounting because of the difficulty and expense in finding the actual depreciation on all units of a large property at short enough intervals of time to make it useful in accounts. If the account were not called "depreciation" this necessity would be removed.

Note.—This Symposium was published in November, 1941, *Proceedings*. Discussion on this Symposium has appeared in *Proceedings*, as follows: November, 1941, by Messrs. Edwin F. Wendt and L. T. Fleming, and Anson Marston.

[17] Cons. Engr., Winter Park, Fla.

[17a] Received by the Secretary November 12, 1941.

The inaccuracy in this method may be illustrated readily. The original Interstate Commerce rules specified a 50-yr life for frame buildings. The writer happens to know of two frame houses in good condition which have been occupied for more than 250 years. A stationary engine or machine is said to have a life of 25 years. The writer remembers an engine used for driving machinery in a roundhouse that operated for more than 70 years. Other instances can be found easily where the actual useful life was less than the theoretical life.

When the accounting concept is removed from consideration, depreciation becomes a loss in value caused by age and wear, changes in the demand on the unit, and obsolescence. The determination of the value of each of these factors and their combination into a percentage which represents the loss in the value of the unit is a matter for trained engineering judgment aided by all pertinent records available.

There has been much discussion, and many books and articles have been written about how to depreciate property when the results are to be used for one of many possible purposes. The general tenor of much of this writing is that the sum used for estimating whether it is economical to replace a unit is entirely different from that to be reported to a commission as depreciation in a rate case. This does not seem to the writer to be good engineering thought although it may appear to be good legal strategy. A decision of the U. S. Supreme Court on April 30, 1934,[18] defined depreciation as follows:

> "Broadly speaking, depreciation is the loss, not restored by current maintenance, which is due to the factors causing ultimate retirement of the property. These factors embrace wear and tear, decay, inadequacy, and obsolescence."

This would seem to be a proper definition. Why should not the engineering profession adopt such a definition, cease to refer to the "straight-line" accounting method as depreciation, use the term "amortized cost" for accounting purposes, and eliminate the idea that the amount of depreciation varies with the purpose for which the data are to be used?

With this done the Joint Committee on Depreciation (under whose auspices the Symposium is presented) could collect, analyze, and publish a great amount of data that would aid the engineer in his judgment when establishing depreciation percentages in any case under consideration.

George E. Goldthwaite,[19] Esq.[19a]—The unwillingness of public utility representatives to face the depreciation problem is reflected in the paper by Mr. Walls. This unwillingness is understandable in view of past history, but it has now become extremely dangerous to public utility interests.

After the close of the "cutthroat" competitive era and before effective regulation had been imposed, most utility owners and many railroads were in the position of possessing monopoly franchises under which profits could be realized far in excess of the present conception of a fair return. Such franchises were

[18] Lindheimer *et al.* vs. Illinois Bell Telephone Co., 292 U. S. 151 (1934), p. 167.

[19] Cons. Engr., New York, N. Y.

[19a] Received by the Secretary November 18, 1941.

considered things of great and permanent value, to which the physical properties were merely attachments, so to speak, which could be readily maintained or replaced without any particular care as to amortization procedure; or in fact without any amortization whatever. The owners did not consider it necessary to face any depreciation problem, and when conditions changed they were naturally reluctant to admit that such a problem existed. Even after the introduction of regulation, the long period of rapid business growth and the era of court-supported reproduction-cost valuations combined to make possible further postponement to a realization of the problem. This past history has colored the views of nearly all utility managements, and has inhibited their representatives from frank statements regarding depreciation.

A capital charge with regard to any item of perishable or depreciable property is merely a deferred operating expense. When a company has spent money for such property and charged it to capital account, it cannot issue an honest statement of earnings for any subsequent year without including in its expenses a due proportion of the cost of that capital item. It is immaterial as to whether it is called amortization, depreciation, or even retirement expense. If it is a due proportion of the cost of the capital asset, it must be based on an estimate of life. It is perfectly futile to argue that a reliable estimate of life cannot be made; an estimate is nevertheless absolutely necessary, because otherwise the earning statement is misleading if it is not actually fraudulent.

As a corollary to the foregoing, the value of a depreciable asset cannot be greater than the amount fairly chargeable against operating expenses within its remaining life. This conclusion also follows from the general premise that the value of a depreciable capital asset resides solely in its capacity for giving service in the future. Without postulating such capacity, there would be no excuse for charging it to capital account. Unless the service capacity is assumed to extend very far into the future with no impairment of efficiency, its value must be largely dependent on the remaining length of life.

The management that bought and installed a piece of electric generating machinery in 1925 had it charged to the capital account in the well founded hope or expectation that it would be useful for a considerable period of future years. None of the officials expected it to last forever; they all knew it would have to be retired within a very limited period of years for some reason or other, and so they should have faced, immediately, the accounting problem of distributing its cost as equitably as possible among the consumers benefited and within the period when it would be useful to them. What excuse did they have for not facing this problem, and how could they purport to issue an honest earning statement without making this effort?

The answer is that they felt relieved from any economic necessity for showing costs because they depended on their monopoly power, supported by the vague phrasing of the court-made law, to enable them to reimburse themselves for their unamortized investment after the asset had disappeared.

They object to the word depreciation because it carries unpleasant implications as to loss in value. Suppose, however, that it is merely called amoritization, and a plan is devised which seems financially sound. If (as the writer has tried to show) the limit to the value of a piece of machinery is the amount

that can be recovered from its output to cover amortization charges during its remaining life, in addition to all other operating expenses and return or profit, then the unamortized cost is inherently an estimate of remaining value. Hence, an intelligently planned amortization program is inherently a depreciation program, and accrued depreciation is best measured by the balance in a properly accrued reserve, subject only to substantial alterations in life expectancy taking place since the program was initiated. Additional value can be realized only through the monopoly power of the utility company, when supported by legal rights, to collect revenues greater than can be economically justified by cost of service.

Professor Grant's excellent paper displays the realistic approach to the depreciation problem that is to be expected from representatives of competitive industry, who have no monopoly power backed by "due process" law to "bail out the water" accumulated by past neglect of financial "leaks." His attack on the spurious plausibility of sinking-fund accounting is good. The practical effect of sinking-fund accounting is to postpone amortization, which is one reason for its popularity in certain circles; but anything that postpones amortization is an invitation to financial suicide.

The writer would like to warn against tying up the replacement problem directly with the depreciation problem. Considerable confusion has resulted from assumptions that a utility plant is maintained in practically 100% condition by periodical replacements of parts as they become worn or obsolete, and from that the false conclusion is reached that the outstanding purpose of depreciation accounting is to build up a fund whereby replacement may be made. Actually the depreciation problem exists with regard to any perishable item of property whether or not it is replaced, and to the same degree. Once a charge has been made to the investment account for the acquisition of a perishable plant item, it is economically necessary to arrange for writing off that investment before the item is retired, and the theoretically correct use of the money is not merely to replace it (because it may never be replaced), but to retire the debt or other obligation incurred in connection with its purchase. It is best, for clear thought on the subject, to avoid as far as possible the conception that the plant item will be replaced by another at the end of its useful life. Actually the replacements are usually indirect. A new power plant is built to take care of additional load; the use of the old is gradually reduced until it becomes only a standby; and finally it is retired and dismantled. There has not necessarily been a direct replacement at any time, but the indirect replacement has been made long before the retirement of the old item. Other examples could be cited in which it is difficult even to find an indirect replacement, such as the discontinuance of service in a certain area due to the removal of industry or shift in population. In any case, it is economically necessary to amortize the investment during the service life, regardless of how or whether replacement is made.

Data available for the statistical study of the service life of a utility property are often of such nature as to justify skepticism as to the results. Nevertheless, numerous statistical studies have been made on a strictly dollar basis, wholly without reference to the physical items (which some may say are quite heteroge-

neous within the given accounting classification), and the writer has obtained results of remarkable consistency as between various properties owned by different companies in widely different areas. It is proper to say that the results should be examined with caution, and should not be used unless they seem plausible by other tests. At the very least, however, they represent, in certain property groups, the best methods now available for statistical study of property life under existing and past conditions. They are certainly better guides to estimates of life expectancy than pure judgment devoid of statistical support.

The paper by Messrs. Crum and Winfrey has so little bearing on the problem of public utility accounting that the writer does not believe it is pertinent to make comment from the regulatory standpoint. If he understands correctly, these authors recommend in effect that a system of accounting which is sound for private enterprise would aid the government in determining the economic status of many of its projects; although there is no direct financial occasion for depreciation accounting on such projects.

Nathan B. Jacobs,[20] M. Am. Soc. C. E.[20a]—Although depreciation is an element that an engineer must consider in the design of any public improvement, it does not necessarily follow that the elements of the depreciation assumption which he uses in selecting a given project are the same as an accountant would consider in setting up annual or accrued depreciation on the books and records of the public authority; nor is either of these to be confused with the depreciation that would be fixed by an appraisal engineer in setting up the deduction which should be made for accrued depreciation in determining present value. In selecting a given project from a group of alternates the engineer computes the probable annual cost, to determine which is the most desirable and economical. In doing this he will use his judgment as to the probable length of service which the project will undergo. If this is a new project the engineer will be guided solely by his judgment, in comparing one project with the others. His judgment, of course, will be tempered by his past experience, but it is his estimate of the future. In accounting for depreciation annually the comptroller will have available and can use the past experience with the particular unit or project, and will continually have the advantage of actual observation to determine how much of the life of the unit, or how much of its value, has gone.

As stated by the authors, the setting up of depreciation in public works accounting and finance is the exception rather than the rule. In fact, clear and careful distinction between capital expenditures and operating or maintenance expenditures is seldom found.

Some years ago officials of the City of Pittsburgh, Pa., desired to exempt, from the limits of municipal indebtedness, bonds which were issued for water-works purposes[21] on the grounds that "the net revenue derived from said property * * * shall have been sufficient to pay interest and sinking fund charges." At that time municipal water-works utilities in the Commonwealth of Pennsyl-

[20] Pres., Morris Knowles, Inc., Pittsburgh, Pa.

[20a] Received by the Secretary November 18, 1941.

[21] Section 15, Article 9, of the Constitution of the Commonwealth of Pennsylvania.

vania were required, by the Public Service Company Law, to keep their records according to a Uniform System of Accounts, which required setting up depreciation as an operating expense. The city controller called the attention of the referee appointed by the court to the fact that if depreciation were set up as an expense, the net revenue would not be sufficient to pay the interest and sinking-fund charges. The referee found:[22]

> "It seems reasonable, therefore, that a depreciation reserve should be set up in some form or other to keep the water plant in a functioning condition and the inclusion of a proper amount for this purpose is a proper item of operating expense. It is also clear that it would not be proper to compel the City in operating a public utility to charge as an item of operating expense both a depreciation reserve and the amount required to meet interest and sinking-fund charges on debt incurred for plant construction. It is immaterial, however, as to the form that a depreciation reserve takes, so long as it is in fact taken care of, and so that the public utility will be continued as a functioning plant * * * since the principal payments for water debt reduction exceeded the requirements for a depreciation reserve * * * no further charge need be made as an operating expense in determining the net revenue of the City from the operation of its water plant * * *."

Although the finding by the referee may be sound practice in public works financing, and the setting up of a reserve in public works, either by the retirement of bonds or by the creation of a sinking-fund, is equivalent to taking depreciation, it is not in accordance with a sound accounting system for a public authority to neglect to charge depreciation, since generally there is no relation whatever between the rates of depreciation and those for bond retirement. The annual debt service for the retirement of bonds or for a sinking-fund as provided by a municipal or public authority in administering its public works will be greater than the amount which an accountant or engineer would place as representing the annual reduction in value which is taking place.

In private business it was always considered to be good policy to "get out of debt" as soon as possible. This is probably more important in public works, where it is not unusual for any given project to be bonded for 100% of its cost, so that it is highly desirable to create an equity for the owner as rapidly as possible. Following this principle, advocates of governmental economy are recommending pay-as-you-go practice in an attempt to get municipalities and others on to a basis where debt will be continually reduced and the item in the budget for debt service can be ultimately eliminated. In many cities today the debt service for interest and retirement or sinking-fund bonds over a period of recent years has been in excess of the amount spent each year on new improvements. The advocates of this pay-as-you-go principle, therefore, are recommending a return to a receipts and expenditures basis, thus eliminating the need for depreciation in municipal accounting.

Although these considerations may indicate that it is not necessary for depreciation actually to be set up on the books for accounting purposes in public works, nevertheless, depreciation must be taken into consideration by engineers in recommending public works, and if a capital account or capital

[22] Hoffman vs. Kline *et al.* 300 Pa. 485.

budget is kept by the public authority, depreciation should be taken into consideration. The use of debt service in lieu of depreciation is dangerous and open to considerable error. In the first place, long-term bonds are often issued in payment of the construction of short-life property, so that the debt service will continue far beyond the physical service of the works constructed. The usual example is the issuance of 30-yr bonds to pay for short-life street paving and re-paving. In public works accounting there is often not a clear distinction between maintenance and replacement. Sometimes bonds are issued for contemplated construction which may not have been done. Recently in one large city 20-yr bonds, issued for subway construction, which subway was never built, were refunded and extended for another period of years. Depreciation should be included in public works accounting and finance, so that proper statements of assets and liabilities can be compiled.

H. J. Flagg,[23] M. Am. Soc. C. E.[23a]—For several years the writer has had the privilege of membership on the Committee on Depreciation of the National Association of Railroad and Utilities Commissioners (NARUC). This Committee is composed of men trained and experienced in one or more of the fields of economics, accountancy, engineering, and management. The function of the Committee has been, and is, to study the various aspects of depreciation. A report is in preparation which will present the results of the Committee's labors. The contacts not only with the members of this Committee but with other students of the subject have disclosed such a variety of opinions that the writer is reminded of the allegorical poem by John Godfrey Saxe entitled, "Blind Men and the Elephant." The poem is too long to quote in full, but its essence may be expressed by abbreviating all verses except the first and the last:

"It was six men of Indostan to learning much inclined
Who went to see the elephant, though all of them were blind,
That each by observation might satisfy his mind.
The first (side) 'is very like a wall';
The second (tusk) 'is very like a spear';
The third (trunk) 'is very like a snake';
The fourth (knee) 'is very like a tree';
The fifth (ear) 'is mighty like a fan';
The sixth (tail) 'is very like a rope';
And so these men of Indostan disputed loud and long,
Each in his own opinion exceeding stiff and strong,
Though each was partly in the right and all were in the wrong."

The writer does not intend to suggest that those in the various fields of professional endeavor who are studying depreciation are either blind or all wrong. The obvious point of the poem is that the nature of the contact an individual has with any subject is certain to influence his views and conclusions. There is a real need for the enlightenment that will flow from an exchange of views such as this Symposium affords.

---

[23] Chf. Engr., Board of Public Utility Commissioners of New Jersey, Newark, N. J.

[23a] Received by the Secretary November 28, 1941.

Mr. Walls has referred to the problem of estimating remaining life of property and has pointed out the forces that both cause depreciation and control remaining life. He correctly states that,

> "All estimates of future life involve uncertainties and are assumptions, and the farther these estimates are extended into the future, the less reliable they become."

Most engineers will readily admit the practical impossibility, in most instances, of forecasting with certainty the remaining life of machines or facilities. With respect to those items of plant that waste away because of wear and tear, corrosion, abrasion, or erosion, there is considerable factual data available which may enable a qualified person to make a reasonable forecast of remaining life in a given situation. The main difficulty arises with respect to those items of plant which are greatly affected by forces whose incidence is unpredictable. It is true that statistical and actuarial methods are in use which undertake to project past experience as a guide into the future. This is done regardless of any question concerning the preponderant and variable influence of unpredictable future events on the retirement of property. These methods appear to give answers which satisfy the estimators, but it may be questioned whether the apparent refinement of the process is worth the cost and effort of the special research that is involved.

One of the projects of the NARUC Committee on Depreciation is to accumulate, with the help of the utility industries concerned, factual data with respect to the causes of retirement of utility property. These causes have been classified in three groups, namely: "Physical causes" such as wear and tear, action of the elements, etc., "functional causes" such as inadequacy and obsolescence, and "other causes" such as orders of public authorities, civic improvements, etc.

It is probably true that different companies will apply the classification differently, although a serious effort has been made to maintain uniformity. Nevertheless it is believed that the diversity of the returns is such that any irregularities in the application of the classification are reasonably well compensated.

Twenty-nine companies for which returns are available retired an aggregate of slightly more than $349,261,000 of plant in the most recent 10-yr period. These companies represent more than $3,300,000,000 in operating fixed capital and more than $678,000,000 in annual operating revenues. Of the total retired it is found that approximately 18.1% was due to physical causes, approximately 72.2% was due to functional causes, and approximately 9.7% was due to other causes. With functional causes having such a dominant influence in the retirement process the difficulty of making dependable estimates of future lives is obvious.

The present-day concepts of depreciation of public utilities appear to have had a slow evolution from the early days when depreciation was not even recognized as a factor in rate making. The present concentration of interest in the subject dates back only to 1934 or 1935. Much progress has been made in a relatively short period, but in view of the intensified attention to the subject,

even greater progress can be expected in the future. It is to be hoped that the time is not far distant when a common basis will be found on which accountants, economists, and engineers can reconcile their presently divergent views. Regardless of what may be accomplished in this direction, it will always be true, as Mr. Walls states, that the treatment of depreciation cannot be reduced "to a simple arithmetical process that will eliminate, or even reduce, the quality or amount of sound judgment required in the treatment of the problem."

Nelson Lee Smith,[24] Esq.[24a]—Referring to the view of Professor Grant, the writer wishes to re-emphasize that much of the confusion that has existed, and still persists, regarding the much-discussed subject of depreciation in relation both to competitive industry and to the public utility field is due to a failure to differentiate sharply between different concepts and to keep clearly in mind the setting of the problem, the purposes of the study, and the uses to which conclusions are to be put.

Mr. Walls has recognized specifically the necessity of this realistic type of approach; and, therefore, he doubtless would agree that, in considering depreciation, regulatory authorities must refer constantly back to the relationships between that phenomenon and the objectives and processes of public control in general. The point which Mr. Walls emphasizes suggests that he is concerned chiefly with depreciation in rate-making and particularly with the accuracy of the deduction made from the rate base on this account. From the viewpoint of the regulatory commission it is equally important that the current charges or accruals for depreciation be determined properly, for they enter directly into the costs, which must be reflected in the rates for service. Since, in a rate case, these two determinations—the rate base deduction and the annual charge—are merely intermediate steps in an integral process (that is, the establishment of rates), it is not surprising that the commissions in their renewed interest in and more concentrated attention upon this problem are emphasizing the proposition that consistency requires the use of the same basic data and assumptions in each.

The regulatory agencies generally would agree with Mr. Walls that depreciation, as he expresses it, cannot be made to suit a definition but, rather, that the definition must truly express the actualities of depreciation; but they would go on to argue that what are alleged to be the facts and realities of depreciation for one step of rate-making must be accepted as having equal validity for other parts of the same process.

Mr. Walls appears to contend that there is a very fundamental difference between the problem of measuring depreciation at any given time and that of estimating future service life. He contrasts the reliability of estimates of the extent to which causes of retirement have already acted with the uncertainties of their operation in the future. At first glance, the difficulty—if it be a real difficulty—of predicting the future will seem to support his position; yet, the distinction cannot be accepted when dealing with depreciation for regulatory

---

[24] Chairman, New Hampshire Public Service Commission, Concord, N. H.

[24a] Received by the Secretary November 26, 1941.

purposes, because this process requires the measurement and expression of depreciation in economic terms.

It may be feasible, through the use of engineering techniques, to determine the extent to which physical deterioration has occurred. At the same time, however, forces of obsolescence may be in operation. Although it may be that the ultimate impact and effect of such factors on service life cannot be foretold exactly, the fact that they are acting is currently and necessarily affecting the total service life, and this is just as important in its relation to the present depreciation of the property as is its mere physical condition.

Of course, this is simply another way of stating that the result can be no more accurate than any one of the factors upon which it depends and, hence, that no greater reliance can be placed on the estimate of the depreciation that has occurred in the past than on the prediction of remaining life in the future.

From his paper, the writer cannot be sure just how much significance is attached by Mr. Walls to the distinction which he seeks to make in this connection. Presumably he does not mean to suggest that data and methods should be used in establishing a basis for the current depreciation charges which are different from those utilized in determining the depreciation already suffered. In such event and for the reasons already suggested, the writer is certain that commissions would be likely to hold that the estimates must be consistent with each other and that greater weight cannot be given to one than to the other. Perhaps Mr. Walls intends, by his emphasis upon this distinction, simply to summarize or emphasize his conclusion that depreciation should not be treated in a superficial or oversimplified manner. With this conclusion the regulatory commissions generally would be in substantial agreement. They, and their representatives on the committee dealing with depreciation for the National Association of Railroad and Utilities Commissioners, certainly concur in the view that the estimated rate for the future should not be a matter merely of calculation based on arbitrary assumptions or fragmentary statistics. They would agree that it should reflect judgment as to what the future is likely to hold. Even so, they would hold that statistical studies of past experience are useful in showing what has happened and what, therefore, other things being equal, may be expected to happen in the future.

If it is proposed by Mr. Walls that such studies are to be rejected or abandoned despite the extent to which they have been utilized voluntarily by utility concerns in their attempts to meet their own problems (which may create some presumption as to their usefulness), a question which may be fairly asked is: What shall be substituted as a means of forming judgments? Is it to be a "medical examination," and if so, what is that "medical examination" to involve?

The element of judgment may enter most appropriately into this process by influencing the determination of the nature and extent of the statistical studies which shall be undertaken and by indicating what studies of this character can be justified. Are group or unit methods better adapted to dealing with particular types of property; and in what manner shall the results, and can the results, properly be applied?

Consequently it appears that the argument for the exercise of judgment is not an argument for substituting that judgment entirely for the statistical approach, but that a better argument can be made for the usefulness of the statistical approach as a means of informing the judgment.

C. Beverley Benson,[25] M. Am. Soc. C. E.[25a]—The principal conclusions to be drawn from Mr. Walls' paper are: (*a*) That he considers depreciation and future life as entirely separate matters rather than that one is, in part at least, a function of the other; (*b*) that the forces which cause depreciation can be measured with reasonable accuracy but that the depreciation they cause cannot be so measured; (*c*) that remaining service life cannot be estimated with any degree of accuracy; and (*d*) that the estimate would be worthless if it could be made. Of course, competitive industry dares not adopt so vague a plan for depreciation; but perhaps utilities, protected in their monopoly position by the courts, can rely upon the regulatory bodies to save them in the event that their depreciation policies prove inadequate.

Granting the difficulties inherent in any objective plan for estimating depreciation, the writer believes that a more definite procedure than the vague plan outlined by Mr. Walls is necessary. Remaining service life is the most important single criterion to be considered in estimating the value of any economic item or group of items. In ordinary activities the amount one will pay for such an item depends upon his conscious or unconscious estimate of the length of time it probably can be used. It is at this point that one finds the distinction between deterioration as an engineering concept of efficiency and depreciation as an economic concept of value. A 4-in. candle will give as much light at the moment as an 8-in. candle of the same diameter, but the value of the 4-in. candle is less because of the fewer hours it can continue to give light. Similarly, improvements to a power plant may improve its coal economy or reduce maintenance expense or improve reliability, but the ultimate value of the improvements is a function of the remaining life of the plant itself. Professor Grant's brilliant paper gives added emphasis to this point, because estimated remaining life forms the basis of much of his extremely interesting treatment.

Although the importance of remaining life as a criterion of value is enormous and (in the writer's opinion) is controlling, it is nonetheless true that the difficulties of estimating remaining life are real and that precision is probably not possible. Therefore, the wise executive will adopt a plan which permits flexibility.

However, there are many reasons for thinking that estimates of remaining life of physical property can be made with reasonable accuracy. In the first place, in a property of any considerable size sudden changes in life expectancy of the group as a whole are not likely, since the inertia of the mass acts as a stabilizer. For example, if a company has 20,000 electric meters and is currently replacing 400 of them per year, it will take a long time for the introduction of a new type to change the average life expectancy of the group.

The objection is often made that changes in the arts cause unpredictable changes in service life; but a properly organized actuarial study contains the

[25] Prin. Statistician (Eng.), New York Public Service Comm., New York, N. Y.

[25a] Received by the Secretary November 21, 1941.

effect of past improvements in the arts. There is little evidence to show that the rate of such improvement varies greatly. Consider, for example, improvements in steam locomotives—the water-tube boiler, the compound cylinder, the outside valve gear, the feed-water heater, higher and higher boiler pressures, superheaters, return to simple cylinders with short cutoff, uniflow boilers, mechanical stokers, etc. Who can say that improvements in locomotives have ended or will continue at an accelerated pace?

It is sometimes said that the effect of obsolescence makes life estimates of little value. It is probably true that the preponderance of retirements of utility property is due to obsolescence and inadequacy, but there is little evidence that these factors are unrelated to age. This point is illustrated by Fig. 1. If the retirement rate is constant (that is, no greater tendency to

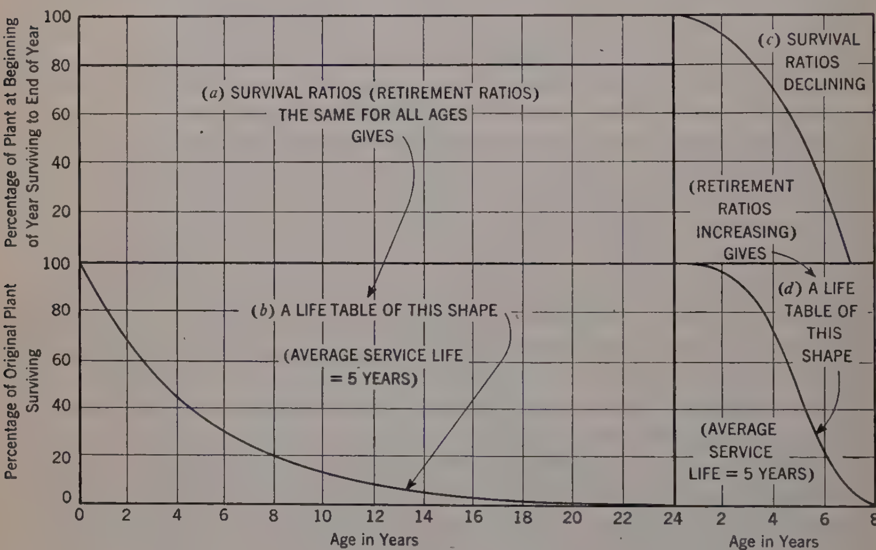
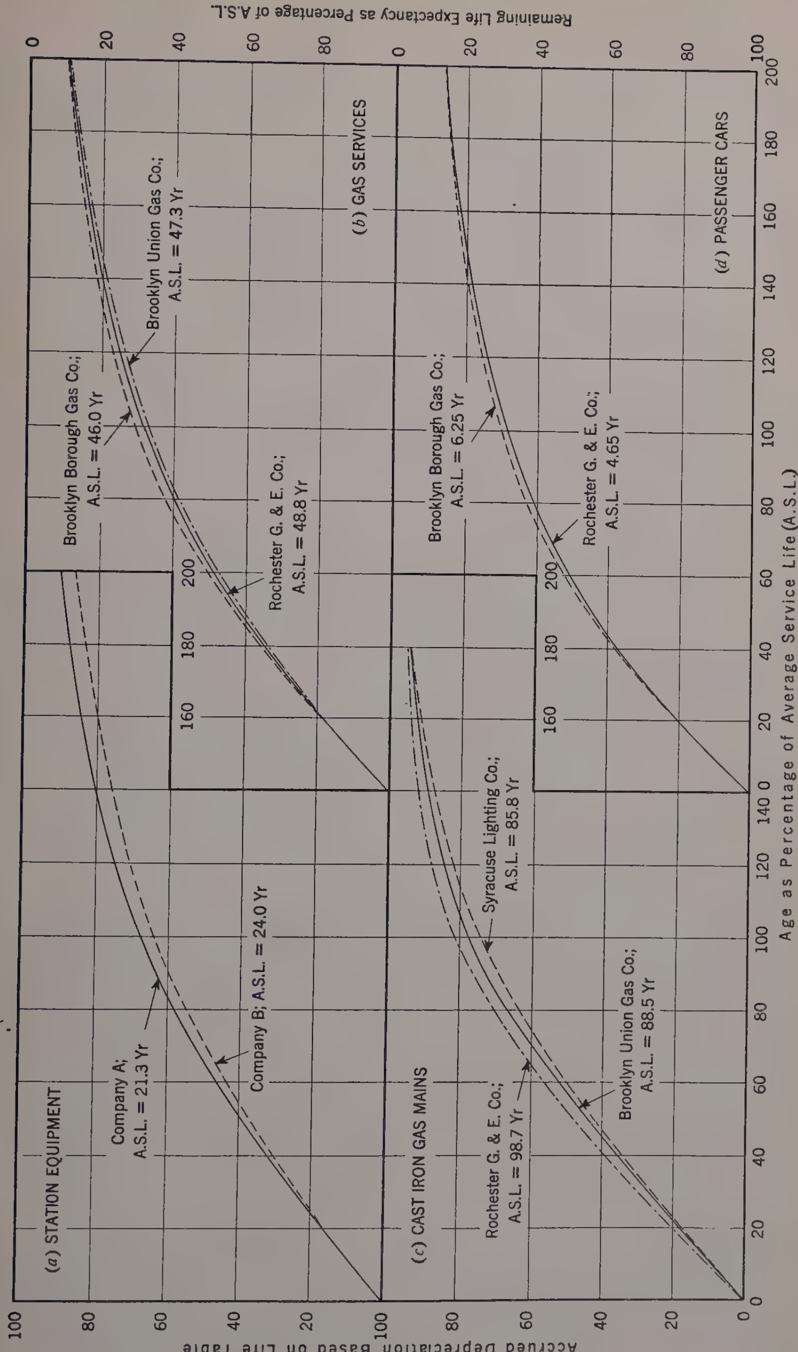


Fig. 1

retirement at one age than at another), the resulting life table will have the shape of a compound discount curve, as shown in Fig. 1(*b*). On the other hand, if retirements are an increasing function of age, the life table will have the shape shown in Fig. 1(*d*).

Engineers of the New York Public Service Commission have constructed more than 300 life tables representing more than $400,000,000 worth of utility property. Since all the life tables they have been able to construct are of the "normal" shape, the conclusion is inescapable that obsolescence is a function of age and, consequently, is no less predictable than many other variable factors.

Another strong reason for having confidence in life estimates lies in the close similarity of results for different companies. Fig. 2 illustrates this similarity for a few groups of property. The most interesting point shown by these graphs is in the similarity of the shapes of the life expectancy curves, although



Remaining Life Expectancy as Percentage of A.S.L.
Accrued Depreciation Based on Life Table
Age as Percentage of Average Service Life (A.S.L.)
(a) STATION EQUIPMENT
Company A; A.S.L. = 21.3 Yr
Company B; A.S.L. = 24.0 Yr
(b) GAS SERVICES
Brooklyn Borough Gas Co.; A.S.L. = 46.0 Yr
Brooklyn Union Gas Co.; A.S.L. = 47.3 Yr
Rochester G. & E. Co.; A.S.L. = 48.8 Yr
(c) CAST IRON GAS MAINS
Rochester G. & E. Co.; A.S.L. = 98.7 Yr
Syracuse Lighting Co.; A.S.L. = 85.8 Yr
Brooklyn Union Gas Co.; A.S.L. = 88.5 Yr
(d) PASSENGER CARS
Brooklyn Borough Gas Co.; A.S.L. = 6.25 Yr
Rochester G. & E. Co.; A.S.L. = 4.65 Yr

FIG. 2

there is considerable variation in the life estimation of similar property for different companies.

However, the writer wishes to emphasize the importance of having such statistical estimates made by engineers. An engineering basis should be used in choosing the type of mathematical analysis; but statistical estimates should be used only as a guide to engineering judgment in connection with a detailed knowledge of the physical plant under consideration.

It is important that actuarial analysis be confined to fairly recent experience, although the width of the experience band will have to vary somewhat with the life expectancy of the type of property being investigated. There seems to be little question that present or very recent conditions should be reflected in any estimate for the immediate future. It must be remembered that the rate at which old property is being retired now is some indication of the rate at which property now young will be retired later.

The cost of statistical analysis of physical property is not prohibitive. Once set up, mortality studies can be kept up to date at very little expense and, with recently developed methods, the cost of the original setup, even for small companies, is not great.

It cannot be denied that regulated industry is in a special position regarding provision for depreciation, since the law requires an attempt to fix rates that will lead to a fair return. Therefore, many of the usual concepts of value in a competitive group must be modified because the ordinary forces of supply and demand cannot act freely. Consequently, a device must be adopted for regulated industry that will tend to protect both owners and consumers from the effects of errors in forecasts. Items once charged in operating expenses must not be charged again if the estimate of life proves too short (if the annual rate is too large). On the other hand, owners must be recompensed in some way if the estimate of life proves too long (if the annual rate is too small). Return must not be allowed on amounts contributed by customers toward ultimate retirements. These aims can best be achieved if the rate base is considered as the total plant less depreciation reserve.

The required flexibility can best be provided through the use of the straight-line group basis for both accrued and annual depreciation. In other words, the calculation of annual charges for depreciation must be based upon the same set of factors as the calculation of the accrued depreciation. Straight-line depreciation on a group basis does not mean equal annual charges in dollars but means a constant annual percentage of surviving dollars. The result of the use of the group basis is that a deficient reserve on property which lasts less than average life expectancy is made up by an excess reserve on property which lasts longer than average life expectancy.

By the use of the fixed capital less depreciation reserve as a rate base, errors in life estimates tend to be offset by inverse errors in return. That is, if the life estimate is too great, the annual charge is too small, the reserve is too small, the rate base is too large, and the owners get a greater return. On the other hand, if the life estimate is too small, the annual charge is too great, the reserve is too great, the rate base is too small, and the owners get less return.

AMERICAN SOCIETY OF CIVIL ENGINEERS

Founded November 5, 1852

DISCUSSIONS

TRAFFIC ENGINEERING AS APPLIED TO RURAL HIGHWAYS

Discussion

BY MESSRS. NATHAN CHERNIACK, AND PARK H. MARTIN

NATHAN CHERNIACK,[7] Assoc. M. Am. Soc. C. E.[7a]—Mr. Harris' paper serves to re-emphasize this point: That in the modernization of existing highway facilities, and in the design of new ones, there is a need for the engineer to adopt a "demand" point of view. This means that the engineer should regard a highway, first, as a facility for regular use by human beings operating machines over its surface for the purpose of traveling between given origins and destinations safely, expeditiously, comfortably, and conveniently; and secondly, as an engineering structure designed to carry specified loads and constructed with definite structural materials. A highway that cannot, at once, satisfy the "demand" specifications is a poor engineering structure, and serves constantly to reveal to the traveling public a disparaging reflection of engineering ability, irrespective of the fact that the engineer may not be to blame for conditions encountered.

Mr. Harris touches only lightly on the subject of economic justification of highway facilities. Where he does, however, he reflects the orthodox theory of economic justification—namely, that reduced highway and vehicle operating costs justify the magnitude of a highway improvement. This theory can stand drastic revision. For example, under "Case No. 3.—Prescription for a Dangerous Route" he states:

> "The average loss of time over this two-mile section was calculated from the data to be 1.08 min or a total over a year's time of 2,222,000 min. * * * Using for this example the value of one cent per lost minute, the total loss capitalized at 5% would possibly warrant the expenditure of $444,400 for improvement."

There seems to be something basically wrong with a statement that a saving of a minute by each vehicle might support a $445,000 improvement.

NOTE.—This paper by Milton Harris, Assoc. M. Am. Soc. C. E., was published in May, 1941, *Proceedings*. Discussion on this paper has appeared in *Proceedings*, as follows: September, 1941, by Charles E. Conover, M. Am. Soc. C. E.; and October, 1941, by Stephen E. Butterfield, Assoc. M. Am. Soc. C. E.

[7] Economist, The Port of New York Authority, New York, N. Y.

[7a] Received by the Secretary November 21, 1941.

In the case of a truck, a minute saved does not necessarily mean a penny or two "saved." There is no saving, for example, in the portion representing fixed costs (taxes, insurance, etc.). These items have been paid for, in advance, and there are no rebates. Truck time saved does have value, but only if such time is substantial and can be utilized effectively. What could a truck operator do with a minute saved?

On the other hand, passenger-car time costs much less than truck time. In fact, in most cases it has no commercial cost whatever. Then again, car operating costs may sometimes prove to be higher, because of greater road distances and higher speeds on improved or new highways than on alternate older routes. All of these facts would tend to reduce the expenditures that could be justified for improved or new highways. Despite these facts, however, the experiences of toll highway facilities indicate that, as between a modern vehicular tunnel with a higher toll and alternate antiquated ferries, for example, or as between a modern toll bridge or toll highway and a congested "free" bridge or highway, a substantial group of passenger-car motorists are paying something extra, in tolls, for the privelege of using superior facilities.

A realistic approach to highway economics would therefore recognize these facts: (1) That operators of trucks and buses are more interested in highway economies that would permit them to make more revenue trips (and incidentally increase operating costs) rather than in those that promise unrealistic "paper" reductions in operating costs; (2) that passenger-car motorists (especially around urban areas) are more interested in reducing the "wear and tear" on their nerves than on their cars; and (3) that well-designed highway facilities can offer travelers positive "new" values, such as speed (consistent with safety), comfort, convenience, recreation, pleasure, and (again, especially in urban areas) freedom from the irritations and annoyances of the older hazardous and congested alternate highways controlled by traffic lights. In studies of the economic justification of modern highways, anticipated time savings may be used as indexes of these "new" values. Usually, however, they are largely neglected because they are not readily measurable.

To determine these "new" values, which, in many cases, overshadow doubtful anticipated operating cost reductions, requires the measurement of differences in the nervous energy used in traversing existing traffic-light-controlled streets and highways, on the one hand, and parkways, expressways, and freeways, on the other. Possibly this may be accomplished by means of portable instruments indicating the differences in the blood pressure under which drivers operate under different highway conditions. As Mr. Harris states in his "Introduction," "Traffic psychology is complex and as yet unsolved. * * * This type of research should be undertaken." Such research apparently may not come within the field of engineering or economics, but it would be highly fruitful and would throw a flood of light on the economic aspects of highway design.

Park H. Martin,[8] M. Am. Soc. C. E.[8a]—Engineering terminology should be specific, and not subject to several interpretations. The use of the word

[8] Planning Engr., Allegheny County Planning Commission, Pittsburgh, Pa.

[8a] Received by the Secretary November 19, 1941

"rural" in the title and body of this paper has caused the writer to speculate at some length as to what the author intended to imply. After careful study, the writer concluded that the term "rural" as used embraced any and all types of highways lying outside of, or beyond, urban centers.

Expression of opinion was sought by the writer from half a dozen engineers of experience in highway work, as to their interpretation of the term "rural highway." Without exception the opinion expressed was that rural highways meant highways extending through rural areas, with very limited vehicular travel.

It seems ridiculous that a term used so much in highway engineering should have so indefinite a meaning. In general, "rural" has been used as defining a certain class of highways. Highway classification terms are not uniform; nor do the words used by one department mean the same when used by another. Several illustrations picked at random follow: From the State of Oregon System—

System	Types
R–1	Rural arterials
R–2	Rural general use laterals
R–3	Rural land service ways
U–1	Urban arterials
U–2	Urban general use laterals
U–3	Urban land service ways

The 1940 report of the State Wide Planning Survey of the Maryland State Road Commission states on page XVII, "This plan for centralization of authority and responsibility contemplates the grouping of all public rural roads into three systems for administration purposes: (*a*) State Primary Road System, (*b*) State Secondary Road System, (*c*) Local or Land Service Roads."

In the State of Pennsylvania a somewhat different classification exists. In 1911, by an Act of the Legislature, a State System of Highways was created. Main roads connecting county seats, cities, and boroughs were taken over as state highways. About 12,000 miles of highway were involved. In 1931, by an additional Legislature enactment, 20,000 miles of highway were added to the system. The 20,000 miles of new highway was designated as the "Rural Route System," the original 12,000 miles being commonly called the "Primary System." The justification given for taking over 20,000 miles of new highway was "To take the farmer out of the mud." Since farmers live in rural areas the term "rural" was given to the addition. Many of the roads taken over were not rural in traffic or service and are less so now, but the term still applies. Conversely many were rural in character and service, and still remain so.

Traffic data made public by the Pennsylvania Department of Highways from an eighteen-county traffic study involving 8,584 miles of road are as follows:

Vehicles per day	Miles of road
<50	1,447
50–99	1,247
100–149	837
150–199	564

The foregoing shows that 4,095 miles of the 8,584 miles investigated carry less than 200 vehicles per day, and the remainder, of course, greater traffic.

The writer is of the opinion that the term "rural highway" is not a proper engineering designation and that highway engineers should have a uniform classification nomenclature to designate the several different classes of highways. The basis for determining the terminology to be applied is debatable. The writer ventures to recommend that traffic volumes and traffic use be the foundation for the classification and terminology. This recommendation is made notwithstanding the opinion of some highway engineers that traffic records cannot be used to divide or classify highway systems. For example, M. R. Keefe, M. Am. Soc. C. E., has written:[9]

> "On studying our traffic records it soon becomes apparent that one cannot divide the existing system into existing primary and secondary roads on the basis of traffic. On many roads one section will be carrying light traffic, 100 to 150 vehicles per day, while on another adjoining section the count may run 750 to 1,000 per day."

The author implies that rural traffic is different from city or urban traffic. The large majority of drivers on rural highways (the term hereinafter used in the largest sense) are urban residents and drivers. The average driving speed is undoubtedly higher, but otherwise the driving habits are not much different on rural highways than upon urban highways and streets.

The author seems a bit inconsistent in his discussion of traffic behavior. Quoting from the "Introduction":

> "By analyzing traffic action, traffic behavior can be determined and evaluated. Roads should be built accordingly; hence, it seems reasonable to expect a road design that will accommodate that behavior which is normal to those who drive with prudence."

In the next paragraph, however, he states:

> "Doubtless, driving habits can be changed by education and by enforcement, but more probably can be accomplished by natural regulation resulting from properly designed highways which accommodate themselves to reasonable existing traffic patterns * * *."

It is true that drivers as a class do have certain uniform habits. For instance, they will not drive close to a curb on an open highway, very many spiralize curves of their own accord, but the engineer must plan from sound engineering principles and not from drivers' changing whims or habits.

The most outstanding project of modern highway engineering with which the writer is familiar and in which has been incorporated every known principle of approved highway design, is the Pennsylvania Turnpike. Yet, notwithstanding the care in planning and expenditures made, drivers will "crack up." It is evident that a wholly foolproof road can never be designed and built.

Four-lane divided highways are given as an example of engineering that will eliminate head-on collision. Several years ago the Pennsylvania State Department of Highways rebuilt a part of the Bigelow Boulevard in the City of Pittsburgh as a freeway with 12-ft lanes, a division strip being protected by

[9] "The Indiana Secondary Road Program," by M. R. Keefe, *American Highway*, October, 1936.

curbs with a 10-in. reveal. Experience has shown that it is a rather common occurrence for cars to crash over the curb and wreck themselves on light standards set in the dividing space. The irony of one of the wrecks was that it was a police squad car. This is not to recommend that the divider should be eliminated, but rather to re-emphasize the personal element in driving.

The author illustrates and mentions the use of photography as an aid in traffic studies. This suggestion is excellent and its use is desirable in various ways, one of which could be the study, by means of moving pictures, of merging traffic behavior.

Another commendable point made by the author is the observation of traffic under varying conditions of light and weather. The writer has long been impressed by the failure of traffic engineers and designing engineers to give more consideration to night visibility on a highway. Traffic islands in many instances serve a useful purpose, particularly when lighted. On rural highways, they may become points of hazard, due to poor visibility at night. Attempts to direct traffic by channelization which involve complicated openings and confusion points should be avoided, particularly when such points are not lighted.

At another place the traffic engineer, or some maintenance crew, may have failed in the placing of directional and speed signs. This criticism refers particularly to older rural highways of poor alinement. The writer has observed many signs that are visible in daylight driving, but are either invisible or only partly visible at night. This is generally the result of improper placing.

The writer is of the opinion that the placement of traffic signs should be studied in the field at night, as well as by day. The proper "signing" of highways, particularly those routes which have poor alinement, and for which funds are not available for reconstruction, is highly important.

The night accident record on rural highways is very bad, as testified[10] by Kirk M. Reid, of the General Electric Company:

> "What are the facts as to the relative day and night hazards on our rural highways? The statistics gathered by your departments show that from three-quarters to four-fifths of the total 24 hour traffic is by day, and yet almost half the non-fatal accidents and over half the fatal accidents occur during the period of dusk and darkness. Hence the average accident hazard per vehicle mile at night is three to four times that by day. And the fatality rate per accident is 64 percent greater at night."

A study of the proper placing of signs that are visible for night driving seems to be a field which needs more attention from traffic engineers. The author has discussed this subject, somewhat, in his example of study involving a curve reduction illustrated in Fig. 6. The result is the recommendation that the curve be reduced with a radius of 644 ft for a speed of 50 miles per hr. This is all very good, but it should be remembered that there are thousands of such curves which probably cannot be corrected because of lack of funds. Why not place a directional curve sign at the proper place at the curve approach and mark on the sign the safe speed for the curve, such as "Speed, 35 Miles"? The distance that the sign should be placed from the point of curvature should

[10] "The Night Accident Hazard," by Kirk M. Reid, *American Highways*, January, 1936, p. 16.

not be less than the distance required to brake the speed of the approaching car from what would be the safe speed of the car in the tangent down to the safe speed recommended for the curve. There is also one additional factor to be considered in setting the safe speed to negotiate a curve: On a two-lane highway the speed should not exceed the distance required to bring a car to a stop if an obstruction, such as a stalled car, is on the road in the path of the moving car.

In the case of the example cited in Fig. 6, the safe sight distance will be less than 250 ft and the safe speed should be less than 50 miles per hr for the corrected curve. The critical speed is sometimes assumed to be about 50% greater than the safe operating speed.

The importance of traffic data for design cannot be stressed too much. Many designers give no thought to this important relationship, if left to themselves. The writer cannot see how there can be a rational design of a highway without all the known facts of traffic being considered. The writer has in mind one project in which the designers proceeded without traffic figures. When those in charge of design were asked for their traffic data, it was found that they had not considered any. A complete origin and destination survey was made and the original preliminary plans were discarded entirely.

The peak-hour traffic is undoubtedly the criterion for which the designer plans. Daily volumes are interesting and of value in presenting the economics of a problem, but the peak-hour load is the controlling factor in design.

Forecasting is interesting but sometimes rather hazardous. Notwithstanding the uncertainties sometimes involved, it must be done. Forecasting the future of a highway may sometimes result in recommendations for a new highway rather than heavy expenditures for the reconstruction of an existing highway. For example, state Route No. 19 is the main route leading north from Pittsburgh. Outside the city limits the road passes, for four miles, through rather populated suburban areas. On one particular section the alinement is poor. For some time there has been considerable agitation to correct this condition by the construction of a bridge. This would entail an expenditure of several million dollars resulting in a considerably depressing effect upon adjacent property values. The construction of a new four-lane divided highway through undeveloped farming land about one mile east was decided to be the better plan. The construction of the new highway (at a cost only slightly more than the cost of the proposed bridge) affords two routes for some distance north of the city, with considerable relief to the existing traffic load on Route No. 19.

The lack of adequate parallel roads, particularly detour routes, in times of construction and repairs, is becoming more apparent and more serious as traffic volumes increase. Minor roads in the vicinity of main routes should always be considered in the light of available detour routes.

The writer cannot agree with the author when he states (see "Traffic Data for Design")—"To obtain actual data for design speed, it is necessary to know what speeds are prevalent over the present route." Maximum speed limits on rural highways are fixed by law. They may vary from 45 to 55 miles per hr. The vast majority of drivers, even under the best conditions, cannot drive safely at a speed greater than 60 miles per hr. It is assumed that the case the

author has in mind involves the reconstruction of the highway. Since the highway is to be modernized, it would seem to the writer that the design should provide for the safe legal speed, which in turn would permit a higher critical speed. Unless the highway is a freeway, the design speed should not exceed 60 miles per hr. Since internal-stream friction (the conflict of faster and slower traffic in the same direction) is the cause of the greatest percentage of accidents, it does not seem wise that greater speeds than legal limits should be encouraged.

Minimum sight distances for passing and non-passing are now well determined and accepted in standard practice. The writer, personally, will remain a critic of three-lane highways. Economically, as well as from the standpoint of safety, they do not appear sound. The only exception that may be noted is in the case of truck lanes in mountainous areas. Joseph Barnett, M. Am. Soc. C. E., has presented[11] an interesting and instructive treatment of highway intersections. When funds are not available for separation devices this phase of rural highways is highly important.

Too much stress cannot be placed on the angle of visibility, not only at intersections but at accelerating traffic lanes. The writer has not been convinced of the desirability of expecting a driver to attempt to look over his left shoulder at a very flat angle in merging into high-speed traffic from an accelerating lane.

In 1941 the City of Pittsburgh, County of Allegheny, and Pennsylvania State Department of Highways completed a grade separation in the city of a street and Bigelow Boulevard. A merging traffic lane is brought into the Boulevard by a curve, the deflection angle of the access lane with the Boulevard being 25°. The radius of the curve between the access lane and the Boulevard is 588 ft. Speed on the Boulevard is about 45 miles per hr. Those who have observed its operation have been very much impressed by the speed and ease with which the traffic merges.

The writer agrees with the author (see "Traffic Data for Design") in that: "The angle of intersection has a direct bearing on design in that the angle of visibility of the driver is confined within relatively narrow limits from within his compartment."

In presenting the subject of relationship of traffic engineering to highway design, the author has performed a service for highway engineers. The writer is hopeful that his discussion of highway classification terminology may result in some terminology more acceptable than the word "rural."

---

[11] "Highway Intersections, Design and Application," by Joseph Barnett, *American Highways*, January, 1940, p. 18.

AMERICAN SOCIETY OF CIVIL ENGINEERS

Founded November 5, 1852

DISCUSSIONS

DESIGN AND CONSTRUCTION OF SAN GABRIEL DAM NO. 1

Discussion

BY WILLIAM P. CREAGER, M. AM. SOC. C. E.

WILLIAM P. CREAGER,[4] M. AM. SOC. C. E.[4a]—For his able description of the tests, design, and construction of this notable dam, the author is to be congratulated. The consulting engineers on this work (L. F. Harza, M. Am. Soc. C. E., and the writer) and the chief engineer (Mr. Howell) advocated a somewhat smaller section for this dam, the finally adopted section being a compromise between their ideas and those of the state engineer's consultants. This point is indicated in view of the steeper slopes adopted for subsequent designs of dams of similar type.

The writer cannot agree with Mr. Baumann's method of stepping up to the prototype the results of tests of the three models as shown in Fig. 24; nor can he agree with the statement (see "The Spillway") that the discharge for all models and probably also for the prototype can be expressed by Eq. 5, having an exponent of 1.535.

In the three models and the prototype, consider, respectively, homologous heads of 0.03, 0.15, 0.75, and 15.00 ft for the scale ratios of 1 : 500, 1 : 100, 1 : 20, and 1 : 1 (prototype). The test head of 0.03 ft in the 1 : 500 model, of course, is entirely too small for consideration on account of the relatively great influence of skin friction. The test head of 0.75 ft in the 1 : 20 model should closely simulate the action of the prototype since, for that size, the influence of skin friction, with a properly constructed model, is negligible.

Beginning with a head of 0.75 ft in the 1 : 20 model and a flow Q of 2.475 cu ft per sec, and assuming that the true exponent is 1.5:

$$Q = 3.81\, H^{1.5} \quad \text{.........................(13)}$$

If this parabola were plotted in Fig. 24 it would give $Q = 221.0$ for the homologous head of 15.0 ft in the 1 : 1 prototype instead of 246.0 determined by the author.

Similarly, the corresponding homologous discharge for the homologous head of 0.03 ft, in the 1 : 500 model, is $Q = 0.0198$ instead of the measured discharge of $Q = 0.0177$. The lower measured discharge is accounted for by the relatively greater effect of skin friction.

NOTE.—This paper by Paul Baumann, M. Am. Soc. C. E., was published in September, 1941, *Proceedings*. Discussion on this paper has appeared in *Proceedings*, as follows: November, 1941, by E. Soucek, Jun. Am. Soc. C. E.

[4] Cons. Engr., Buffalo, N. Y.

[4a] Received by the Secretary October 30, 1941.

AMERICAN SOCIETY OF CIVIL ENGINEERS

Founded November 5, 1852

DISCUSSIONS

FORT PECK SLIDE

Discussion

By Irving B. Crosby, Affiliate Am. Soc. C. E.

Irving B. Crosby,[22] Affiliate Am. Soc. C. E. (by letter).[22a]—The causes of the Fort Peck slide have been discussed by Mr. Middlebrooks in his very useful paper, and he has shown that the slide was due to weakness in the foundation shale and not to any weakness in the dam itself. The writer believes, however, that certain geologic factors have not been brought out by the author and that an erroneous impression has been given by some of the statements.

The Board of Consultants unanimously agreed that the slide "was due to the fact that the shearing resistance of the weathered shale and bentonite seams in the foundation was insufficient to withstand the shearing force to which the foundation was subjected,"[23] but as to the relative importance of the various factors the Board could not reach agreement. The writer is solely responsible for the following ideas, some of which may be at variance with those of other members of the Board:

The Bearpaw shale in the vicinity of Fort Peck Dam has been intersected by numerous faults or fractures on which movement has taken place. Many of these faults were due to deep-seated causes, but some of them were caused by landslides in the bluffs of the Missouri Valley at a time when the river was flowing at a lower level. The principal results of the faults were the production of numerous planes of weakness and of channels for ground-water movement. Study of the numerous boring cores and records made after the slide indicated that there was horizontal movement on a bed of bentonite and that this plane of movement reached the surface largely through weathered shale. The writer believes that the bentonite beds and faults played a very important part in causing this and previous slides.

Mr. Middlebrooks has described the hydrostatic pressure found in many borings in the shale and has ascribed to this pressure an important part in

Note.—This paper by T. A. Middlebrooks, Assoc. M. Am. Soc. C. E., was published in December, 1940, *Proceedings*. Discussion on this paper has appeared in *Proceedings*, as follows: March, 1941, by Jacob Feld, M. Am. Soc. C. E.; April, 1941, by Joel D. Justin, M. Am. Soc. C. E.; May, 1941, by Messrs. William Gerig, Alfred J. Ryan, and Glennon Gilboy; and September, 1941, by Frank E. Fahlquist, Assoc. M. Am. Soc. C. E.

[22] Cons. Eng. Geologist, Boston, Mass.

[22a] Received by the Secretary November 13, 1941.

[23] "Report on the Slide of a Portion of the Upstream Face of the Fort Peck Dam," Corps of Engrs., U. S. Army, July, 1939, p. 8.

causing the slide. He states that prior to the construction of the dam the bentonite seams were covered by only a shallow overburden, that they had expanded and taken up water, and that then the superimposed load of the dam caused high hydrostatic pressures in the bentonite, which facilitated the slide. No such condition (water taken up by the bentonite) existed, however, in the case of the older slides in the bluffs. In those slides the bentonite was still under a thick load, and it did not have opportunity to take up water and expand. These slides were due to undercutting of the bluff by the river with consequent indirect and very gradual overloading. Similar slides on a small scale have occurred in the banks of the reservoir since the water was raised and lowered, and in every case observed by the writer there was a bed of bentonite. Similar slides occurred at Snake Butte, Mont., associated with beds of bentonite in the shale, but in this case the peculiar situation of thin overburden and sudden overload assumed by the author did not exist and yet slides occurred.

The writer believes that the water squeezed out of the shale in the various borings came largely from the intricate network of fractures in the shale and to only a small extent from the pores of the shale and bentonite, and that "liquefaction" of the bentonite previous to the slide did not exist. Once movement occurred, however, water from the fractures may have been mixed with the bentonite and facilitated the extent of movement. The presence of flat bentonite beds and of fault planes inclined toward the valley provided planes of low resistance on which movement occurred when the superimposed load became sufficiently great.

Mr. Middlebrooks concludes that the best means of avoiding similar slides in the future "is to always obtain representative undisturbed samples of this type of rock, sufficiently large to reveal the true character of the rock to geologists and engineers." Numerous undisturbed samples of this shale and bentonite were taken, however, and numerous tests were made on them prior to the slide; but the true condition of the shale was not understood and the slide was not foreseen.

This writer doubts that more and larger samples would have disclosed the dangerous foundation condition unless the geologic history of the region had been correctly interpreted first. The history of past slides is written physiographically on the face of the bluffs. Nature has been conducting large-scale tests on the shale, over a long period of time, in a manner that cannot be duplicated in the laboratory. Correct understanding of these natural tests will often reveal what laboratory tests fail to show. The tests, by nature, in the bluffs have shown that the bentonite is the weakest natural material in this region and that slides frequently occur on it. Correct interpretation of the physiography and stratigraphy shows that great masses of shale have slid from their original positions and that the shale in the bluffs may be intersected by innumerable zones of weakness.

With the geologic and physiographic history of the region correctly interpreted, the possibility that a dangerous situation may exist becomes apparent. Then clues that may have been overlooked would be recognized and warning given, and the actual conditions could be disclosed by special investigations. Had the true conditions been understood, the dam could have been designed to meet those conditions safely.

AMERICAN SOCIETY OF CIVIL ENGINEERS

Founded November 5, 1852

DISCUSSIONS

VALUE OF PUBLIC WORKS'

Discussion

By J. P. Hallihan, M. Am. Soc. C. E.

J. P. Hallihan,[31] M. Am. Soc. C. E. (by letter).[31a]—The writer is gratified that his attempt to make an analysis of the value of public works has met with such generous discussion. That was his hope, as it is evident that, when an emergency problem of this character is presented, an analysis of the actual results obtained under experimental processes and the troubles encountered in the pursuit of a relief works program will be of the greatest possible aid in determining the possibility of future betterment in methods. For all practical purposes, the paper might just as well have been entitled "The Value of Time," since the results indicate that lack of preparedness was a very important factor in delaying the process of getting the program of public works under way.

The analysis of the situation in the United States as a whole, under the conditions of stress and disturbance of the national economy that were made evident in 1930, required much the same collection of facts as would be presented to a receiver of any large private property that suddenly found itself unable to meet its current necessities. For example, one becoming unexpectedly heir to a great ranch property in a run-down condition would find his primary consideration directed to an inventory of his assets—the condition of the ranch in regard to summer and winter feed, the water supply and its possible distribution to bring it within easy reach of the grazing herds, the creation of adequate reserves against long droughts, and the question of whether the cattle stocked were of breeds best suited to the ranch conditions. All would be basic situations to be considered before entering upon a program of rehabilitation of the property.

Similarly, there existed in 1930 only very sketchy information on the condition of the natural resources of the United States, its forests, lands, water re-

Note.—This paper by J. P. Hallihan, M. Am. Soc. C. E., was published in February, 1941, *Proceedings*. Discussion on this paper has appeared in *Proceedings*, as follows: April, 1941, by Clarence W. Post, M. Am. Soc. C. E.; May, 1941, by Messrs. Uel Stephens, William J. Wilgus, Bernard L. Weiner, Albert Ed. Scheible, H. B. Cooley, and Philip W. Henry; June, 1941, by Baxter L. Brown, M. Am. Soc. C. E.; and September, 1941, by Messrs. Evan S. Martin, Isador W. Mendelsohn, and W. W. Crosby.

[31] Director, Municipal Eng. Section, WPA, Washington, D. C.

[31a] Received by the Secretary November 8, 1941.

sources, and in particular its mineral resources in the light of the possible demands of national defense. Progress has since been made toward catching up on this deficiency. In many cases it has been found that great losses have been sustained by failure to have reasonably complete information available and to determine whether properties previously regarded as assets were still in that category or were in an advanced stage of depletion requiring long periods of conservation and building up processes before they could again be considered as assets.

The writer did not intend to convey the impression that the development of a public works program was the only thing necessary to do in combating an industrial depression. The record shows that it proved to be the most readily available measure to provide the necessary employment quickly and to secure the greatest salvage in the creation of needed facilities against practically the same expenditure on direct relief pending what was thought by many to be the certain revival of industry if let alone.

In the period beginning in 1930, no effort of consequence was made by the vast industrial machine to set up a program that would take the place of a federal-aid public works program. The privately owned industries are not to be blamed for reluctance to gamble their reserves, which would certainly be needed to rebuild obsolete plants on the resumption of industrial activity, in the expenditure of such sums before the trend of recovery became clearly defined. The same hesitancy and doubt caused the hoarding of private capital rather than expenditure in private construction, such as housing, not wholly dependent upon industrial recovery but facing the imperative necessity of taking up obsolescence. In this field the deficiency was so great that it would have required something like three times the volume of public construction to approximate the same relative volume of total construction. In the face of this condition, the writer is of the opinion that the program of public works so painfully developed was the only measure that had any effect in stopping the descending spiral caused by the combined effects of acceleration of unemployment and diminishing purchasing power. In considering its beneficial effects, the collateral employment created in the industries supplying materials and in the service industries should not be overlooked. The United States as a whole perhaps may be grateful at some future date that attention was thus brusquely called to its own deficiencies and that through the experience of corrective efforts and mistakes a plan may be devised that will find the Nation, in the event of recurrence of similar conditions, with a comprehensive plan of operations that will make it able to meet the same kind of emergency with measures adequately prepared in advance of need.

The writer is impressed with the comments of Mr. Henry in comparing the depression of 1893 with the depression of 1930 to 1939, and the thought that the quick recovery from 1893 to 1895 (which was followed by the silver campaign depression, equally quick in recovery, and a continued long period of prosperity to 1907, a temporary money panic) could have been duplicated in 1930. In 1893, the United States was still only partly developed, and the opportunity to initiate individual enterprises and to develop lands, mining, and transportation offered quite a different set of conditions to those confronting

the country beginning in 1930, when the depression was affected not only by the conditions in the United States but by world-wide conditions.

In a chart[32] prepared by Brig.-Gen. Leonard P. Ayres the line of normal conditions shows depressions at various intervals, dropping in 1893 to a maximum of 20 points below the economic line (which was the greatest drop since 1808) and to 26 points below normal in the primary postwar depression of 1921. What is termed by General Ayres as a secondary postwar depression, commencing in 1930, fell in 1932 to a low of 48 points below normal and, after a brief temporary rise above normal in 1937, dropped again in 1938 to 36 points below the normal line, from which it ascended to 4 points above normal at the end of 1939, dropping again in 1940 to 14 points below, and is now (1941) again on the road up due to the defense boom.

General conditions in the United States were also different in 1893. It was not considered unusual for men to move from a place of unemployment to a place of employment where they worked in camps, and to follow the harvest. The country was in the process of terminating the work on its great trunk line railroads and feeders, where the method of operation was to draw raw labor from the congested centers, transport it to the job, and train it on the job to do the work required. That was the same kind of operation that has been required in the six years 1935 to 1941 under the relief works program, except that the relief program was obliged to find work at the point where the excessive unemployment existed. The writer believes, therefore, that the possibilities of early recovery that were present in 1893 did not exist in 1930. Conditions had become too crystallized. The temper of the people was less resilient. Time was lost from 1930 to 1933 by waiting for some evidence of the spontaneous recovery indicated by Mr. Henry, and it appears doubtful that in any future similar situation it would be logical to expect any such recovery by private initiative under the conditions now prevailing.

In Captain Mendelsohn's discussion the point is made in paragraph 7 that the practice of the PWA in contracting the work on public works financed by loan and grant of federal funds produced a greater proportion of off-site employment than the WPA methods, leaving the inference that the indirect payroll through high mechanization was of greater importance than the direct payroll in relieving the unemployment and in increasing purchasing power. The writer is unable to agree with this view as the direct payroll produces an almost immediate effect in circulation through the channels of trade, whereas there is a considerable lag in the final distribution of the indirect payroll. The quantity of materials purchased for the same type of structure, of course, would be the same, regardless of the method of construction.

In Table 5, Mr. Mendelsohn presents the ratio of PWA orders for structural materials to the total production of such materials, indicating the beneficial effect on the suppliers by reason of such orders. The effect would be more clearly shown had he included the substantial volume of orders for materials placed by the WPA in the same period. Purchases of principal construction materials by the WPA and sponsors (not including purchases by other federal

[32] "Annual Business Activity Since 1790," by Leonard P. Ayres, 14th Ed., October, 1940.

agencies financed by allocation of WPA funds) for the period from July 1, 1935, to December 31, 1940, aggregated $987,563,052.

The writer does not consider it within the scope of the paper to discuss questions that have been posed concerning the comparative cost of work done under contract methods and under the limitations of the laws governing the operation of relief agencies. One statement may be made, however, toward clarifying the thought on this subject. The cost of a public works job performed by contract methods, where the contractor may mechanize the operation to its fullest extent and exercise unrestricted freedom of selection of the cream of the labor from the general labor pool, does not include one important element of cost, which is the cost of maintenance, in condition ready to serve, of the general labor pool. The objective of the laws governing the operation of relief agencies in aiding public works projects with federal funds is that primary consideration should be given to the greatest possible employment of labor. In this operation, the proportion of cost attaching to the project for the maintenance of the labor pool is measurably included. It has been made clear in this emergency that this cost is a fundamental element that has heretofore been sustained by the entire Nation. It may be best that it should be so sustained because of the obvious difficulty in designing a method to apportion the collateral costs equitably to the particular work involved. It has been made equally clear that no equitable comparison between the cost of contract work and the cost of work performed under the relief program can be made without consideration of this factor.

The writer expresses his appreciation of the high quality of the comments and papers presented in the discussion and invites attention to the creation on June 24, 1941, by the Federal Works Agency, in cooperation with the National Resources Planning Board, of a division of planning entitled the Public Works Reserve, which presents great potentialities as a measure of preparedness.

The primary purpose of this division is stated to be the promotion of comprehensive studies of the situation in municipalities preliminary to the planning of improvements covering at least a 6-yr period with general plans and preliminary estimates of over-all cost, and the selection from such plans of projects deemed most necessary by the communities concerned for inclusion in a tentative construction program. Thus such projects may be ready for immediate consideration with respect to the financing and development of working plans for contract bidding or assignment of labor forces promptly upon the coming of emergency conditions.

It is apparent that in such studies any deficiencies in statutory or charter legislation will be brought to light and measures for correction prepared for action by the appropriate legislative bodies. These studies and plans involve no financial commitments but are designed to present an adequate picture of possibilities and to clear the field for action by reducing to the lowest point all legal and procedural retarding elements.

AMERICAN SOCIETY OF CIVIL ENGINEERS
Founded November 5, 1852

DISCUSSIONS

CONCRETE IN SEA WATER: A REVISED VIEWPOINT NEEDED

Discussion

By Homer M. Hadley, Assoc. M. Am. Soc. C. E.

Homer M. Hadley,[34] Assoc. M. Am. Soc. C. E. (by letter).[34a]—It is the privilege of the writer to answer, or to attempt to answer, the objections that have been raised to his paper. On the moot subject of concrete in sea water there would probably have been a wider discussion and more objections raised if times were less troubled and more leisurely than now. However, a sufficient amount of skepticism has been presented, directly, indirectly, and by reference, to stimulate the writer to further and closing endeavors, as follows:

First be it said that the paper attempted to give a summarized but complete statement of facts and conditions as they exist; then an interpretation of those conditions. As to the statement that there are a large number of concrete structures of considerable age in sea water along the Pacific Coast, which are free from deterioration, no denials were entered. On the contrary Messrs. Stanton, Way, and Paxson confirm and indorse these statements, and Mr. Squire, although favoring the chemical attack theory, specifically declares that the substructure of the Ferry Building at San Francisco, "approaching its 50th anniversary," is free from deterioration. Therefore, the existence of the numerous old and unaffected structures, not having been challenged or denied, may be accepted as reality by every one. This may be done the more readily because beyond any one's assertion, pro or con, are the structures themselves, standing in place and giving their own testimony to whoever examines them. Mr. Squire appears to feel that conclusions based on a mere quarter century's performance are too hasty and precipitate and that good behavior for the first 25 years is no assurance as to what may happen in an ominous future; but apart from his warning no dissent on the widespread immunity from sea-water

Note.—This paper by Homer M. Hadley, Assoc. M. Am. Soc. C. E., was published in January, 1941, *Proceedings*. Discussion on this paper has appeared in *Proceedings*, as follows: March, 1941, by Thomas E. Stanton, M. Am. Soc. C. E.; April, 1941, by Messrs. W. F. Way, and Glenn S. Paxson; May, 1941, by Messrs. Lester C. Hammond, Ladis H. Csanyi, and G. M. Williams; June, 1941, by Messrs. Harry E. Squire, and J. W. B. Blackman; October, 1941, by Messrs. E. C. Jack, and Alfred M. Freudenthal; and November, 1941, by T. B. Rights, Assoc. M. Am. Soc. C. E.

[34] Regional Structural Engr., Portland Cement Assn., Seattle, Wash.

[34a] Received by the Secretary October 27, 1941.

attack has been expressed. It is largely in the interpretation of the deterioration which has occurred that variance of opinion is most pronounced.

Before coming to the other matters, however, the question of the significance of these old and unaffected structures may well be asked. If sea water attacks them why are they immune? If they are immune—and they are—what is the matter with the sea-water attack? Where is it? Why doesn't it attack? In short, where is the reality—in the structures or in the theory that they contradict and refute? Under these conditions the theory manifestly becomes untenable. There is no satisfaction in retaining and cherishing a theory that does not work, however plausible it may be. A prediction of future occurrences must be duly fulfilled if faith and credence are to be maintained in the prediction. Therefore, it appears advisable to the writer that the inevitable sea-water attack theory be abandoned because of altogether too much conflicting evidence. However, should any one be minded like Mr. Squire to retain it for another 10 or 20 years, let him feel wholly free to do so. Time is not an end. Nevertheless, he should bear in mind what was said of old: "Hope deferred maketh the heart sick."

Also deserving of fullest consideration are the partly deteriorated structures—those which combine disintegrated parts with perfectly sound and unaffected parts. Why do they do this? How can such things be? If the sulfates of magnesium, or what-have-you, attack the concrete in some places, why not in others? Attention is directed to Figs. 2, 4, and 6 as food for thought for every one, but particularly for those who yearn after the chemical and the obscure.

It is gratifying to find that there is general agreement on the need for proper mixtures, minimum water content, careful placement, etc. Since the importance of these matters is now widely recognized, it follows that concrete in sea water will have a far better record in the future than it has had in the past.

The dearth of supporting laboratory data in the paper receives adverse comment in two discussions. That such data is most conventional is admitted. It is orthodox, regular, "almost always done." Yet is there need for it as an aid in permitting one to believe the existence of that with which he is confronted? Assuming that the observer stands before concrete that has been exposed to sea water for many years and that he not only sees that it is sound and unaffected but on striking it with a rock or a hammer he likewise feels that it is hard and unyielding and hears it ring to the blow as only hard, sound concrete does, then on this evidence of three of his five senses—all that are involved—it is submitted that without more ado and without waiting for core borings he may well believe that it is indeed unaffected by the sea water. Such may seem a daring, risky, hazardous thing to do on one's own initiative and "without benefit of clergy," so to speak, but inasmuch as there is a wealth of corroborative experience to be had, the act is recommended. If there were not the additional and supporting evidence, physical and chemical analyses would assuredly be most desirable. An extraordinary anomaly deserves the fullest investigation; but this is no rare thing, this unaffected concrete. It is everywhere, even constituting the major part of badly deteriorated structures.

Relevant, therefore, appear the words of Lewis Mumford, who can see no need for research to prove that water is wet or that pepper tastes hot on the tongue. There is nothing particularly wrong with such research, of course, but why, laboriously and scientifically, demonstrate the obvious? What is needed far more than any laboratory research or cutting of cores is a comprehensive and detailed survey and report on conditions as they exist, supported with ample photographic evidence. Every one could then form his own conclusions from the photographs—and there would be little to argue about. In general, the cause of sea-water attack is no more than this: Too much water in too lean mixtures frequently assisted by careless placement. Change these conditions and the attack disappears. Retain these conditions and the attack follows. "Pay your money and take your choice."

Several advance the suggestion that where deterioration does occur the sea-water sulfates hasten and accelerate the destruction. Possibly this is true—possibly it isn't. What quantitative data exist to support the hypothesis? For many years the writer has looked at concrete—good, bad, and indifferent—not only in sea water but in fresh water as well, and he can only say that his impression is that the deterioration is substantially alike, allowance being made for the greater water movement and greater mechanical destruction along the seacoasts. Following Mr. Hammond's reference to them, the writer reread the articles[17] by Messrs. Wig and Ferguson published in 1917,[34b] and he looked at the illustrations of these articles. Then he looked through a dozen or so volumes of the *Proceedings* of the American Concrete Institute (A. C. I.), which have carried many a paper and report about durability of concrete, deterioration of concrete, frost-resistant concrete, etc., principally illustrated with examples in the Great Lakes region. For every Wig and Ferguson illustration of sea-water deterioration—reinforcement troubles excepted—he thought he found its counterpart illustration of structures around the Great Lakes, far from the sulfates of magnesium. To any one interested in seeing for himself, these several writings are commended.[35] Also, let every one who would make sea-water resistant concrete read a paper by R. B. Young,[36] published in 1940. It presents the recipe although not written with that thought in mind.

Captain Blackman and Mr. Freudenthal advance the view that the making of proper concrete for sea-water use is a matter of delicacy and extreme refinement. "A hair, perchance, divides the false from true." To which view the writer must dissent. Given reasonable proportions and control there is no cause for believing that a slight departure from the established quantities is fraught with dire consequences. If water is kept to the minimum which placement will permit, and if the cement content is not less than 1.50 bbl per cu yd, considerable fluctuations can occur without ill effects. They are not recommended, of course, but neither should they be appalling to contemplate if they do occur.

[17] *Engineering News-Record*, Vol. 79, Nos. 12, 14, 15, 16, and 17.

[34b] Correction for *Transactions:* In May, 1941, *Proceedings*, p. 919, line 9, change "1935" to "1917."

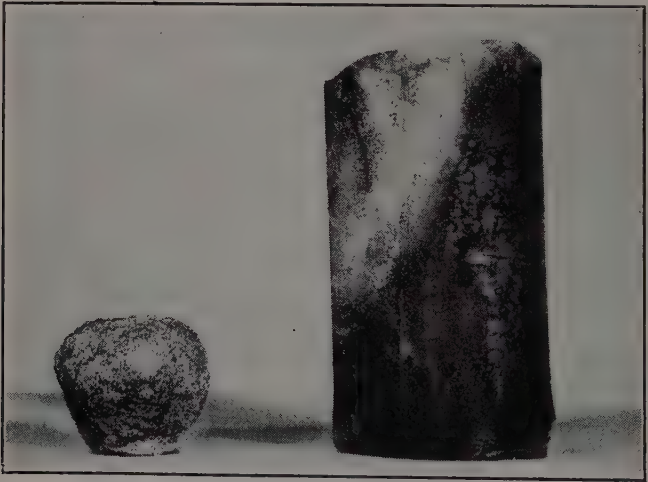
[35] See particularly illustrations in *Proceedings*, A. C. I., 1925, pp. 271–279; 1929, pp. 65 and 66; 1937, pp. 1055 and 1068; and 1940, p. 479.

[36] "Frost Resistant Concrete," by R. B. Young, *loc. cit.*, 1940, p. 477.

Of all the evidence in support of the sea-water attack theory, the writer knows of nothing more convincing than tests described by Mr. Stanton[21] in 1938. He has seen these specimens with his own eyes and accepts the findings without hesitation. There is clearly demonstrated by immersion (not in some

(a) Standard Ottawa Sand; 1 : 3 Mix (b) Standard Ottawa Sand; 1 : 2 Mix (c) Graded Russian River Sand; 1 : 3 Mix

FIG. 9.—VARIATIONS OF MIX; CEMENT SPECIMENS AFTER FOUR YEARS IN SEA WATER (17.2% C_3A)



(a) 17.2% C_3A (b) 7.2% C_3A

FIG. 10.—VARIATIONS OF C_3A-CONTENT; CEMENT SPECIMENS AFTER FOUR YEARS IN SEA WATER (STANDARD OTTAWA CEMENT; 1 : 3 MIX)

[21] "Testing Cement Mortars in Sea Water," by Thomas E. Stanton, M. Am. Soc. C. E., *Engineering News-Record*, March 17, 1938, p. 400.

other alkaline water but by immersion in sea water itself) the fact that pronounced differences in these specimens develop between cements of high and low C_3A-content. Figs. 9 and 10 are views of certain of Mr. Stanton's typical specimens at the age of 4 years. There is no question as to the way in which the test specimens have behaved. However, Fig. 11 of pier 17 on the San Francisco waterfront, built in 1912, is highly relevant to the conclusions that are to be drawn from Mr. Stanton's investigations since it is built of that same brand of cement used in the tests which was found to have 17.2 C_3A-content and to have the poorest resistance to sea water of any of the cements tested. Despite this fact, the piles of pier 17 are unaffected by sea water and the little

Fig. 11.—Precast Concrete Pile Jackets; San Francisco Pier Built in 1918

fins that formed at the joints in the forms still project from their surfaces. Inevitably the question arises: "So what?"

It is to be recognized by every one that the same name on a sack of cement does not mean the same identical cement from year to year unless raw materials and manufacturing processes remain unchanged. However, no change of major consequence has occurred with this particular brand of cement. It uses the same limestone and same clay and essentially the same manufacturing process today as it did in 1911 and 1912. Its raw materials are more carefully proportioned, more finely ground, and are better controlled; the burning is better controlled; the clinker is more finely ground. Beyond these changes,

however, the manufacturing process is substantially unaltered. It can safely be assumed, therefore, that the cement of pier 17 is practically the same as that used in Mr. Stanton's tests. Other differences, therefore, must account for the extraordinary contrasts in behavior. These other differences are immediately apparent in the contrasting mixtures and contrasting specimen structure that results. In the one case a highly permeable porous structure was sought and obtained with mortars using sand of practically uniform particle size. In the case of pier 17 a concrete mixture with its aggregate particles grading from fine to coarse was used.

There are other differences also—those of shape, form, and size. A 2-in. by 4-in. cylinder, although a very convenient laboratory specimen, resembles nothing that is built in structures; on the contrary, it is quite unlike anything so built. How great these differences are is shown in Table 1, in which are to

TABLE 1.—RELATIVE SHAPE AND SIZE OF CONCRETE SPECIMENS

Description	Volume (cu in.) (1)	Surface (sq in.) (2)	Edge length (in.) (3)	Curvature[a] (4)	Col. 2 / Col. 1 (5)	Col. 3 / Col. 1 (6)
2-in. by 4-in. cylinder	12.57	31.416	12.57	57.30	2.50	1.00
4-in. section of 16-in. round pile	804.04	201.06	0	7.16	0.25	0
Ratio, 2-in. by 4-in. cylinder to pile	.....	......	....	8	10	∞

[a] Curvature of edge and side, in degrees per inch.

be noted particularly the curvature (Col. 4), and surface-to-volume (Col. 5) and edge-to-volume (Col. 6) ratios. The little cylinder is much more unfavorably shaped than is the corresponding round pile or square pile section. The square pile has more edge length—usually chamfered—but has flat plane surfaces.

There is not only pier 17 at San Francisco in conflict with Mr. Stanton's and others' findings regarding C_3A-percentages. Mr. Squire records the 11% C_3A of the unaffected San Francisco Ferry Building substructure, which has been nearly 50 years in sea water. The cylinder substructure of pier 38 at San Francisco built in 1909, the base of the lighthouse on the San Pedro breakwater at Los Angeles built in 1910, the cylinder shells of pier 8, Puget Sound Navy Yard at Bremerton, built in 1911 are all 30 or more years old, are all perfectly sound and unaffected by sea water, although the cylinders of pier 38 contain numerous honeycombed areas dating from their original construction. They were built of that brand of cement which in Mr. Stanton's tests was found to have 13.2% C_3A-content. Average analyses of bin samples of this cement for the years 1909 to 1913 ranged from a low of 14.5% to a high of 15.9% C_3A-content. Moreover Mr. Stanton advises that his specimens made with high C_3A cement but with the graded Russian River sand still showed no signs of attack after seven years of the same complete immersion in sea water that proved so disastrous to the specimens made with Ottawa sand. It is for these reasons that the writer is unable to attach more than minor significance to the

C_3A-content of cement for sea-water use. It certainly is important in 2-in. by 4-in. cylinders made of Ottawa sand; but what further claims are (by evidence) warranted concerning it? The disparity between 2-in. by 4-in. cylinders and ordinary piles is great, but how much greater is the disparity between 2-in. by 4-in. cylinders and 4-ft cylinders, 8-ft cylinders, small bridge piers, large bridge piers! The differences may become of wholly different orders of magnitude. The matter of concern is the behavior of the structure and the prototype. Therefore distinctions which, by a process of magnification, are made manifest in a laboratory appear to the writer to have little significance unless they are also found in the prototype; and if they reveal no effect in the prototype, what practical importance attaches to them?

Mr. Way's detailed account of experiences with a large new concrete pile job and of what was found on examination of a near-by pile structure, 15 years old, is a valuable one, particularly in its bearing on the importance of transverse cracks in concrete piles. Frequently a great deal of alarm, agitation, and trouble to all concerned result from fine shrinkage cracks in piles which, if the concrete is of proper quality, cause no harm whatsoever. It is not true that the mere admission of sea water through fine cracks to the reinforcement inevitably brings about progressive rusting of that reinforcement and disruption of the enclosing concrete. If the concrete is (or was) of a lean, wet mix, such very probably will be the result; if the concrete was not lean and not wet, nothing of the sort will happen. A few miles west of Olympia, Wash., in 1920 a small concrete bridge was built across the head of Mud Bay, the southernmost tip of Puget Sound. The columns of the bents were poured in place. At high tide they are almost completely immersed in sea water. At low tide they are completely bare. At a number of places on the square, chamfered corners of these columns, drift logs have broken off the concrete, exposing the main vertical reinforcing bars for several inches. These exposed lengths, of course, are heavily rusted; but no progressive rusting lengthwise along the bars or splitting of the concrete has ever resulted. The rust formed to the edges of the concrete and stopped there. The writer makes this statement with the greater assurance because he broke off some of the abutting concrete, further exposing the reinforcement, and saw that the rust stopped at the edge of the original fracture. Why should this be, if the admission of sea water through the least fine shrinkage crack means doom and perdition? Well, it means no such thing.

An interesting case showing the effect of too much water is in the San Francisco district where considerable corner cracking over the main longitudinal bars occurred in a large pile job, necessitating rather difficult and extensive repairs. The piles, as is almost always the case, were cast in a horizontal position. Three of their sides were thus poured against forms; their top sides were troweled. When the cracks developed, in practically every instance they occurred on the troweled side of the pile—that is, on what had been the top side. "Water gain"—a fine film of water on the under side of the top longitudinal bars, which water film later left an equally fine void space wherein rust could form and progress—appears to be the explanation of this case. Bond tests by H. J. Gilkey, M. Am. Soc. C. E., S. J. Chamberlin, Assoc. M. Am.

Soc. C. E., and R. W. Beal[37] and by Carl A. Menzel[38] show strikingly the effect of casting position—"orientation." It is also worth recording here that the very unfortunate and widely celebrated early experiences with concrete piles in the work of the Los Angeles Harbor Commission were had with piles of 1 : 2 : 4 mix, some of which at least (as photographs attest) were cured by a man sprinkling them with a hoze; how continuously or how intermittently is not revealed by the photographs.

Mr. Paxson's comments on sea-water concrete along the Oregon coast are appreciated. His observation that concrete in deep water shows less deterioration than that on the shore where abrasive and scouring action occurs is highly significant and amusing as well: The more completely the sulfates of magnesium surround the concrete the freer it is from deterioration.

To Mr. Hammond the writer would refer his preceding discussion of Mr. Stanton's tests. He would question Mr. Hammond's statement that Messrs. Wig and Ferguson "demonstrated" that chemical action was noted all along the Pacific Coast. Their papers published in 1917 reveal that they found a considerable amount of deteriorated concrete. The interpretation they placed upon it was that the deterioration resulted from sulfate attack. Presumably they examined all of the old concrete structures shown or referred to by the writer since they state[39] that their examination had been made in the preceding 2 years and that they had made "personal examination of nearly every concrete [marine] structure * * * in the United States." Nevertheless, having reread their papers, the writer cannot see that they "demonstrated" sulfate attack. They attributed the deterioration to sulfate attack. It would be very interesting, however, to see how well their theory of immunity to sea-water attack achieved by the formation of lime carbonate near the outer surfaces of the concrete would be sustained by little 2-in. by 4-in. Ottawa sand specimens that were given proper initial curing followed by generous air curing, and what, under these circumstances, the effect of varying C_3A would be.

Mr. Csanyi's discussion, reporting the condition survey of nineteen structures in New York Harbor, is very interesting. Regarding the continuously submerged parts of concrete structures that cannot readily be examined, it is to be said that little or no question has ever been raised about them. It appears to be the general experience that this concrete does not deteriorate. When the piers of the Seaside Bridge of the Union Pacific Railroad were removed from Cerritos Channel, Los Angeles Harbor, in 1934, after 26 years of immersion, they were found to be wholly unaffected. They were built of a $1\frac{1}{2}$: 3 : 5 mix, not 1 : 3 : 5 as so frequently and so unfortunately has been done in the past. "Not a bad spot in them," declared the superintendent of Merritt-Chapman-Scott Corporation who took them out. The concrete fragments from the piers certainly confirmed this statement. When the old bridge at the site of the new Purdy Bridge, Pierce County, Washington,[40] was taken out, its foundations had been in sea water for 15 years. One of the piers had to be

[37] "Bond Between Concrete and Steel," by H. J. Gilkey, S. J. Chamberlin, and R. W. Beal, *Bulletin No. 147*, Eng. Experiment Station, Iowa State College, Ames, Iowa, 1940.

[38] *Proceedings*, A. C. I., 1939, p. 517.

[39] *Engineering News-Record*, September 20, 1917, p. 532.

[40] *Engineering News-Record*, March 3, 1938.

removed completely. Its concrete throughout was found to be in perfect condition and its reinforcement clean and without rust.

The rigor of Mr. Csanyi's refutation of the writer's contention is lessoned and abated by the fact that his studies, experiments, and conclusions are based not on behavior of test specimens in sea water but on their behavior in a 43% solution of Epsom salts. It would appear to the writer that they are highly relevant to concrete that is to be given such exposure, but that the relationship of the 43% Epsom salts solution to sea water has not been established. Pending that determination the writer will continue "in the error of his ways," if such it be.

Professor Williams' brief discussion is charged with numerous doubts. Regarding chemical analyses, the subsequent discussion of two subjects referred to by Mr. Squire will be pertinent. As to Professor Williams' statement that "Rich concrete mixtures may fail by cracking without prior indication of surface swelling, spalling, or disruptions," the writer would inquire if he (Professor Williams) has ever seen this happen with rich concrete mixtures, not in other alkaline waters, but in sea water. If he has, it is to be hoped that full particulars may be reported, for assuredly it is a highly unusual, not to say unique, occurrence, the like of which has not come under the writer's observation. The terms dense and impermeable in the paper were not used academically but as they are commonly employed. The writer disclaims any responsibility for establishing the criteria by which the sulfate attack of sea water upon concrete is supposed to evidence itself. Messrs. Atwood and Johnson[2] quoted E. Candlot who attributes the sea-water sulfate attack theory and its supposed manifestations to L. J. Vicat. The writer merely assumed (see heading "The Prevailing Viewpoint") that if the theory were valid the evidences of the attack should be discoverable. The paper purports to be no more than an unequivocal statement that such evidence is not to be found along the Pacific Coast and a report of what deterioration is to be found. Also, it was stated at the outset: "* * * it is with sea water in its normal range of concentrations—not with other sulfate waters—that the paper is concerned" (see "Introduction"). It was not without knowledge of much to the contrary that has been published earlier that this paper was written. Despite that knowledge, it was written, quite deliberately.

Mr. Squire raises a number of points. One of these concerns the San Pedro breakwater specimens referred to in the paper, the cutting of cores from which, and the saturated condition of the interior concrete of which, on the basis of his "visual" and therefore, by his stern standards, very questionable inspection only, are attested to by Mr. Squire. What the result of the Bureau of Standards' tests and analyses might be was naturally an interesting question. The blocks gave every outward evidence of being all right but were they so in full reality? "Gilded tombs do worms enfold." One of Mr. Squire's fellow citizens in the Bay region who had read the magazine article cited[10] wrote to the U. S. Engineer office at Los Angeles and duly received a reply concerning the

---

[2] "The Disintegration of Cement in Sea Water," by William G. Atwood, M. Am. Soc. C. E., and Arthur A. Johnson, *Transactions*, Am. Soc. C. E., Vol. LXXXVII (1924), p. 205.

[10] *Western Construction News*, June 25, 1932, p. 367.

findings. When the writer made personal inquiry at that office, he was shown a copy of this letter, which was quite in accord with other information he had received. There was nothing about the findings to make them of a confidential nature, and there appears to be no good reason why they should not be summarized herein from notes made at the time.

The Bureau of Standards felt that the tests were inconclusive, principally because the cores were of small diameter (3 in.). Tests were made of eleven cores—five of 1 : 2 : 4 mix and 6 of 1 : 3 : 6 mix. Six brands of cement had been used—three European and three American. The maximum size of aggregate was 2 in. The cores were sawed into cylinders, 3 in. high, which were first tested for strength and then analyzed chemically. Strengths varied widely. Values with the 1 : 2 : 4 mix ranged from 6,650 to 3,110 lb per sq in. Values with the 1 : 3 : 6 mix ranged from 7,560 to 1,630 lb per sq in. No signs of disintegration were noted in the broken cylinders. The report pointed out how the low strengths and erratic values were readily accounted for by the varying position and size of the coarse aggregate particles in the cores. It did not attempt to explain how, after 27 years in the tidal zone surrounded by sea-water sulfates, the high values found could occur. Chemical tests were made with specimens of five of the six cements, two of which were in 1 : 2 : 4 mix cores, the other three being in 1 : 3 : 6 mix cores. There were six samples of the former and nine of the latter. The tests were to determine percentages of magnesia, sulfuric anhydride, and chlorine. Chemical analyses were stated to be inconclusive because of inability to secure representative samples from the small quantity of material in the cores. The samples of 1 : 2 : 4 concrete showed a 74% increase in magnesia content accompanied by a 19% decrease in SO_3-content, which decrease is not in good form chemically when concrete is disintegrating. The samples of 1 : 3 : 6 concrete showed a 358% increase in MgO accompanied by a 51% increase in SO_3. In considering the numerical magnitude of these percentages it must not be overlooked that they are percentages of original very small quantities. Regarding the samples of two of the three cements used in the 1 : 3 : 6 mix specimens the report stated: "While B–1 and B–4 would seem to indicate that some magnesia from sea water has replaced part of the original lime of the cement, this fact may be solely one due to the question of sampling. There apparently has been no increase in the SO_3 content, which is what should accompany this disintegration." The view was expressed (let borers of cores and takers of small samples give heed) that 2 to 3 cu ft of material were needed to make a satisfactory sample.

Inasmuch as there was no definite evidence of physical or chemical deterioration—which is entirely in accord with the many structures referred to—the writer feels quite warranted in stating that there was "no evidence of sulfate of magnesium attack." He feels further confidence in "what is going on under the outer skin of concrete" if this "outer skin" itself withstands scrutiny.

Mr. Squire introduces evidence of concrete in sea water in Europe by reference to Russian tests in the waters of the Baltic Sea at Libau, together with a choice quotation from the report[27] concerning them: "* * * slowly perhaps,

[27] *Proceedings*, International Assoc. for Testing Materials, 6th Cong., New York, N. Y., 1912, Second Section, XVII.

but none the less surely" the processes of deterioration proceed, etc., presumably demonstrating that the San Pedro blocks were "an almost exact replica" of the Russian blocks. Thanks to the reference, the report on the tests was obtained and read. The highlights of, and additional quotations from, it are as follows:

Tests were made in June and July, 1905. Eight test blocks—four masonry and four concrete—were sunk in 1891 and 1892. The concrete blocks measured 6 ft by 6 ft by 10 ft. The concrete mix, by volume, measured separately, was: 1.0 Portland cement, 2.5 sand, 1.5 gravel, and 4.6 "rubble stone"—that is, 1 part of cement to 8.6 parts aggregate, measured separately. "Rubble stone" was stated to be granite "in pieces 10 to 14 cubic inches"—in other words $2\frac{1}{8}$-in. to $2\frac{1}{2}$-in. cubes. Quoting:

> "The greater part of the Portland cement came from Russian Works, foreign cement was also used, but no chemical analysis was made. * * * The sand was taken from the sea on the spot."

Concrete blocks Nos. 8, 9, and 10 were immersed with about $4\frac{1}{2}$ ft of water above their tops. The fourth, No. 15, was set on the breakwater with its bottom at water surface. Incidentally, the Baltic is tideless, and the inflow of fresh water from numerous rivers exceeds evaporation:

> "* * * hence an enormous surplus must be got rid of by an outflowing current which is named the 'Baltic stream' * * *. The fresher Baltic water can be traced far into the North sea especially along the Norway coasts even as far as North Cape."[41]

On removal from the water the blocks were found to be "in good state of preservation" but with various defects: "White exudations," "liquefied mortar," "white liquid derived from deteriorated mortar" in seams and cavities, etc.

> "As a whole the three cements used, slag, Portland, and quartzose cements, were found practically equal in quality."
>
> "The physical tests having given but indifferent results in regard to the determination of the change which the mortar undergoes in the blocks, a chemical analysis of the various kinds of mortar was made.
>
> "The analysis made afforded bases for the following conclusions: If after the determination of the principal elements * * * of the mortar tested and after elimination of the elements proceeding from the Libau sand, the results be compared with the elements forming the cements used (Portland, slag, and quartzose cements) it is easy to ascertain the extent to which the elements forming the cements have undergone variation by action of the seawater.
>
> "* * * The transformation was characterized principally by the freeing of the lime * * * and by the absorption of the oxide of magnesia and of the sulphuric acid contained in the seawater" (refer to Le Chatelier, Michaelis, Feret *et al*).
>
> "The same tables show that the calculated proportion in weight of alumina and iron oxide exceeds in some instances the proportion of the same elements which were in the cements used in making these mortars. This is probably the case because the average composition of the Libau sand reckoned upon does not always compare completely with the composition of the sand actually used. The Libau sand has in fact but little

[41] *The Encyclopædia Britannica*, 14th Ed., 1929, p. 1016.

homogeneity. The chemical analyses of five samples showed the amount in weight of Al_2O_3 and Fe_2O_3 to vary within very wide limits viz. from 0.10 to 0.70 percent. This is due to the presence of particles of earth and clay; these are found especially when the sand is taken in quarries on the downs and not at the sea shore.

"It should be remarked also that although in the mortar made with quartzose cement the quantity of Portland cement was half that in the mortar made with Portland cement, the chemical analyses of the mortar made with quartzose cement was not inferior to that of the Portland cement one.

"One fact, nevertheless, was certain—that the process of deterioration had already commenced and that it progressed, slowly perhaps, but none the less surely in the case of the mortars considered."

There appears to be little need for comment on the preceding testimony. The specimens were made 50 years ago, and the control conditions were not those regarded as necessary today; but as an instance of the highly extolled European practise and experience the reference has its interest. One further matter has its interest also. The English magazine "Port and Harbour Authority," for June, 1941, reprinted the present paper in its entirety with editorial comment. Some of this editorial comment was to the effect that here was surprising news: Concrete in sea water in America that had not deteriorated! True, it did not generally deteriorate in sea water in Britain, but practically all they had ever heard of sea-water concrete in America was that it deteriorated and "went to pot." Apparently while they were receiving such information from this side of the Atlantic, Americans in turn were hearing of the superior and long-established European practise that would not think of using Portland cement in sea water without treatment or process of some kind or other. What in current slang is called "the run-around" seems to have prevailed over a long term of years.

After so much disagreement with Mr. Squire, it is a pleasure to concur with him about the case of the Ford Motor Company Plant at Long Beach (except in minor detail) and to again express warm and sincere admiration for Mr. Russell's work in testing in connection with it. This is the case the writer referred to under the heading "Unsound Materials." It is undoubtedly the one Captain Blackman refers to in his discussion. It appears to be shown conclusively that the felspathic aggregate was the cause of the trouble. The concrete used in the piles deserves Mr. Squire's characterization as "good concrete" in every respect, except that all unwittingly the unsound aggregate was incorporated in it. Thereupon it ceased to be "good." As for Mr. Squire's "chemical reactions caused by the sea water," the writer would gladly subscribe to them while directing attention to Mr. Russell's demonstration, previously cited, of the "chemical reactions caused by the sea water" upon this aggregate all alone and without any cement present while the reactions were in progress. Since the aggregate alone is most unstable in the presence of sea water, it seems scarcely necessary to speculate upon the effects of the addition of cement. It is a not uninteresting fact that while this trouble was occurring at Long Beach the Ford plants at Richmond, Calif. (on San Francisco Bay), and at Seattle, Wash. (in the strongly saline waters at the mouth of the Duwamish

River), built at the same time as the Long Beach plant, were—and still are—free from any trouble in their concrete piles.

So far as the writer knows concrete piles are impregnated with asphalt primarily to protect the pile from low tide to its top, rather than to "protect the concrete under water." This latter protection is incidental to the manufacturing process. If this belief is correct, it is probably more fitting to so state.

With his friends, Captain Blackman and Mr. Squire, the writer has had some stirring arguments in the past. On this occasion may he differ with Captain Blackman regarding the number of Portland cement structures north of San Francisco by saying that, although most large structures are in the south, there is a wealth of lesser structures in the north—bridge piers, pile trestles, sea walls, culverts, outfall sewers, etc., together with a considerable number of larger works. It is not necessary to build a cathedral to have fine architecture reveal itself. Neither is it necessary to have a 1,000-ft long pier to determine the behavior of concrete in sea water.

The writer is made happy by Captain Blackman's statement that deterioration of concrete in sea water by the action of sulfate of magnesium is generally considered as secondary by engineers engaged in harbor work. It is hoped that Captain Blackman reads Mr. Way's discussion, particularly those parts dealing with the concrete pile jobs, cracks, etc. It is believed that the little bridge near Olympia, with certain parts of its reinforcing steel exposed, may also yield him cheer should the hazards to which unimpregnated concrete piles are subject weigh him down unduly.

He refers to the 3-in. cover over reinforcement recommended by the Portland Cement Association[29] and more recently by the 1940 Joint Committee Report.[42] Although this is needed cover with some kinds of concrete, it is not an unmixed blessing, as Captain Blackman knows full well, particularly as it affects the weight of a slender reinforced member subject to bending both in installation and in subsequent service. The writer is of the opinion that the suggestion of J. H. Wasson[43] advanced in 1929 that the tops of piles coming above low tide be made of a particularly good quality of concrete would warrant the omission of extra cover. As a matter of fact, it is doubtful in view of the performance of the 1918 ships and barges, with their limited cover over reinforcement, whether with richer mixes 3 in. of cover is not a wholly unnecessary precaution. Furthermore it would appear that the newly developed vacuum process might have very valuable application in the field of concrete piles in improving the density of the surface concrete and in permitting the reduction of cover over reinforcing bars to 1 or $1\frac{1}{2}$ in.

Captain Blackman delicately refers to special sulfate resisting cements. He states that "The chemists of cement companies have been endeavoring to find a remedy for attacks on cement concrete exposed to sea water and sea air." Undoubtedly this is, or has been, true. Without knowing the exact basis of

[29] "Concrete Piles," Portland Cement Assn., November, 1939, p. 28.

[42] *Proceedings*, Am. Soc. C. E., June, 1940, Pt. 2.

[43] *Proceedings*, A. C. I., 1929, p. 760.

their action, the writer would assume it to result from the plethora of past writings about what is the best European practise, about sulfates of magnesium in sea water, and about all calamitous, inevitable things that befall concrete in sea water. Probably these special cements are highly efficacious in genuinely severe sulfate exposures.

The writer gladly declares that the North Vancouver Ferry Pier is built upon columns, not piles. Fig. 3 is a view of part of its substructure. Considering the diagonal bracing shown in Fig. 3, it is very unlikely that any would think it was built on piles, but if they do, may this statement set them right. The beach beneath the pier is cemented hardpan, and piles could not have been driven.

Considering Captain Blackman's familiarity with the North Vancouver Ferry Pier, it was amazing to the writer to read his statements regarding Burrard Inlet, Vancouver's magnificent landlocked harbor which, with its north arm, covers an area of 31.6 sq miles. True, a number of small streams and creeks or little rivers flow into it from the mountains on the north side. The largest of these, the Capilano River, has its discharge greatly reduced by the use of its water in the water system of Vancouver. Sometimes these streams flood, sometimes they grow small; but at all times, compared with the mighty volume of sea water passing in and out of the harbor with every changing tide (the tides average 12 ft), the inflow of fresh water is insignificant indeed. The harbor authorities report a salinity of harbor water of 1,023 compared with 1,026 for ocean water and 1,000 for fresh water. Consequently "upstream and downstream from the ferry wharf," although a fanciful whimsy, seems more reminiscent of Captain Blackman's old home at New Westminster, British Columbia, on the Fraser River than applicable to Vancouver Harbor. The fact is that, unimpregnated though it be, the North Vancouver Ferry Pier is a very fine, very successful little sea-water structure now aged 32 years. Across the harbor on the city side, where no streams discharge, the Great Northern Pier built in 1913, the Ballantyne Pier[44] built in 1923, the Canadian Pacific Pier B–C,[45] and various other structures are free from deterioration and sulfate attack.

Regarding Mr. Jack's suggestion that sea water may "speed the parting guest," the earlier part of this closure presents the writer's opinion. Mr. Jack's résumé of the various deficiencies of cement is acknowledged as essentially correct, without the writer's having knowledge of how the deficiencies may be overcome. For many uses it would also be desirable to have cement and concrete of half their present weight. To have definite control over the time of set appears to the writer to be particularly desirable for sea-water concrete.

Certain of Mr. Freudenthal's points and objections have been answered previously. His stated experience with aluminous cements in sea water is not confirmed locally. The pier at the Puget Sound Navy Yard, in Bremerton, Wash.,[46] is now (1941) 15 years old, and its cylinders built with high alumina cement are, after 15 years in sea water, in absolutely perfect condition. On the

[44] *Engineering News-Record*, May 24, 1923, p. 908.

[45] *Loc. cit.*, December 16, 1926, p .984.

[46] *Proceedings*, A. C. I., 1927, p. 79.

other hand the writer heard of some bridge in Chesapeake Bay built about the same time with this same type of cement whose piers disintegrated very quickly. Probably this was not in any way due to the sea water but rather to inadequate provision for the dissipation of the large quantities of heat generated in this cement's setting process. Seattle has quite a number of street pavement patches made with high alumina cement. Some of the early ones crumbled and broke up. After it was learned that such patches had to be continuously sprinkled or ponded all trouble ended. It is obvious that sea water had nothing to do with these street pavement failures.

The effect of puzzolanics is speculated upon by Mr. Freudenthal. The south main pier of the Golden Gate Bridge at San Francisco was built with a high-silica puzzolanic cement, the north main pier with ordinary Portland cement. Perhaps some time in the future distinct differences will be observed between them, although, as Mr. Paxson notes, in deep water changes are much slower than at the shore. Bonneville Dam on the Columbia River was built with a puzzolanic cement. It was completed in 1937. The resistance of its concrete to abrasion and cavitation has not been wholly satisfactory, but it is believed that the number of shrinkage cracks in the dam are fewer than if it had been built with Portland cement, a very comforting thought.

Mr. Rights' discussion lacks sufficient detail to permit much comment on the failure of the railroad test piers he describes. The size and shape of these "piers," the cement content of their concrete, and the exact character of the change that is beginning to manifest itself in the remaining pier (described by the words "beginning to disintegrate") are either not stated or are indefinite. The year the test was initiated (1925) suggests the possibility that the "quick-setting high-strength cement" referred to by Mr. Rights may have been the same as that used in the Seattle street pavement patches previously mentioned. This is only conjectural on the writer's part, however. Under these conditions there does not appear to be much basis for comment on the failure referred to by Mr. Rights.

There is no question, however, that frost adds greatly to the severity of the sea-water exposure, and it may be that concrete of a quality that would perform satisfactorily in sea water where freezing conditions are absent would not be satisfactory where freezing conditions prevail. On the other hand, the writer sees no reason for believing that unfaced concrete of a quality adequate to resist the sea-water exposure under freezing conditions cannot be readily obtained.

In 1923 Meyer Hirschthal,[47] M. Am. Soc. C. E., reported sound 16-yr old concrete in a wall at Hoboken, N. J. The North River is not heavily saline, however. Nevertheless it is probably comparable to the estuary cited by Mr. Rights. It would be interesting to know the present condition of this unfaced wall. In his private files the writer has correspondence regarding two test prisms, 12 in. square and about 8 ft long, set in the tidal zone at Saint John, New Brunswick, Canada, in 1913. Freezing conditions prevail at Saint John and the specimens had four square corners, which do not help any toward resisting either abrasion or weathering. In 1933 one of these specimens (made

[47] *Engineering News-Record*, January 17, 1924, p. 127.

of 1 : 3 : 5 mix, gravel aggregate, and "fairly wet mix") was reported to be entirely disintegrated. The mortar surface had disappeared and the specimen was in broken pieces. The other specimen, of 1 : 2 : 4 mix, crushed rock coarse aggregate, and of "about 2–3 slump," was reported to show "only slight wear just about high water."

It is to be hoped that a further report may sometime be made regarding the Aberthaw tests of unfaced concrete specimens placed in sea water in 1909 at the U. S. Navy Yard at Charlestown, Mass. The last published report was made in 1920. In 1940 the writer sought additional information regarding them, learned that twelve specimens of the original twenty-four still remained, and was unable to learn anything more. Since a very good record of the manufacture of these pile specimens exists, it is to be hoped that their condition can be reported. They are now (1941) over 30 years old and have a background of experience that will not be available to test specimens made today until thirty future years have passed.

Granite facings are both admirable and costly. So far as the walls shown in Figs. 2 and 4 are concerned, a solution to their troubles, simpler and less expensive than granite, would have been to have changed the careless construction practises plainly evident in them. There is plenty of unaffected concrete in the tidal zone in both of these views to demonstrate the nature and cause of the deterioration. The wall on the right side of Fig. 2, starting at the stepped-off joint at the reentrant angle is Quay Wall E, built in 1896, and still showing no need of granite facing. The only recent use of granite facing on the Pacific Coast that the writer knows of is the protective course in the tidal zone in the piers of the new Lions Gate Bridge at the entrance to Vancouver Harbor. There is no freezing or ice or unusual condition of abrasion to be concerned about at present, but if the glacial age ever returns these piers are foresightedly prepared to withstand it.

The Dallas Road sea wall at Victoria, listed in the paper as built in 1911, has a belt course of granite 2 or 3 ft high at the beach line, which has undoubtedly been helpful in protecting the concrete from abrasion. These are the only Pacific Coast uses of granite facing on concrete that the writer can think of, although there are quite numerous cases where concrete structures have had granite or other stone blocks dumped loosely around and against them. Granite of good quality, of course, is one of the finest and most resistant materials of construction that there is, and the writer would say nothing to impugn it. As a necessity for the preservation of concrete in the tidal zone, however, he entertains what he at least would call reasonable doubts concerning it. According to reports reaching him, a certain resin either interground with the cement, or introduced into the concrete as an admixture, gives most interesting promise of being a substitute for granite facings when the urge for special treatment in the tidal zone is strong upon one.

To sum up, the situation appears to be this: The sea water exposure is a very severe one. It has been sufficiently demonstrated that concrete can be made which will give a highly unsatisfactory performance in sea water. Undoubtedly bad concrete can be made with any brand of cement and with a wide variety of

admixtures and treatments. The disintegrated concrete is, in general, characterized by a lean, porous, and frequently segregated structure. Too little cement, too much water, too little care in placement are the chief causes of trouble. Poor aggregate is sometimes a further contributing cause. On the other hand there are a large number of concrete structures of considerable age in sea water which are substantially free from deterioration or any evidence of sea-water attack. These have been made with many different cements, differing widely in composition and in percentage of tricalcium aluminate. This immunity occurs with, and is characterized by, concrete of dense, impermeable structure. There is no sea-water deterioration to be found, other than that due to rusting of reinforcement, that is distinctly different from fresh-water deterioration. There is no peculiar sulfate attack. For these reasons it appears to be highly important to use fairly rich mixtures, minimum mixing water consistent with good placement, care in placement, and all reasonable measures which result in density and impermeability. Special safeguards against sulfate attack can be reserved for use where they are needed—that is, elsewhere than in sea water.

To all who contributed discussions the writer would express his thanks and his hope that all differences of view are dispassionate and impersonal. Out of the reporting of various experiences comes increased knowledge and better understanding. It is to be hoped that such may be the outcome here with respect to concrete in sea water.

AMERICAN SOCIETY OF CIVIL ENGINEERS

Founded November 5, 1852

DISCUSSIONS

CRACK PREVENTION PROGRAM, HIWASSEE DAM

Discussion

By O. Laurgaard, M. Am. Soc. C. E.

O. Laurgaard,[7] M. Am. Soc. C. E. (by letter).[7a]—In addition to the formal written discussions of this paper, certain phases have been discussed in conversation and by correspondence with other engineers. The writer was pleased with the favorable comments by all the discussers, because they are familiar with the subject and qualified to express their opinions. In general, he agrees with most of them, but in some instances questions will be answered, the text clarified, or explanations made.

The contribution by Mr. McHenry is interesting and valuable to the general discussion regarding flexible detail specifications for a successful crack prevention program; the use of low-heat cement and temperature studies; the use of thin concrete lifts during construction, based upon theoretical studies of temperature distribution and resulting stresses; the comparative value of the item of low casting temperatures; the relative value of cooling the concrete, in place, artificially by embedded water pipes; and the balancing of the saving in cement against the cost of the crack prevention program.

The writer agrees with Mr. McHenry that the paper would have lost none of its value or interest had it included more of the discussion or reasoning that led to the adoption of certain procedures and the rejection of others, together with a final valuation of the effectiveness of the various items listed in the cost schedule; but such data are so numerous and formidable (as expressed subsequently by Mr. McHenry) that time and space would not permit the inclusion of all of them. The items enumerated in the second paragraph of Mr. McHenry's discussion were considered by the engineers of the TVA in connection with the crack prevention program as adopted.

Although it may be admitted that the pre-cooling of the mixing water was not as effective as had been hoped, the writer cannot agree with Mr. McHenry

Note.—This paper by O. Laurgaard, M. Am. Soc. C. E., was published in March, 1941, *Proceedings*. Discussion on this paper has appeared in *Proceedings*, as follows: September, 1941, by Messrs. Douglas McHenry, Reginald H. Thomson, and W. R. Waugh.

[7] Cons. Engr., Railway Exchange Bldg., Portland, Ore.

[7a] Received by the Secretary November 17, 1941.

that this item was not as justifiable in proportion to cost as the other items. From Table 5 it may be seen that for the five months from May to September, inclusive, in 1939, for shift 1, the air had an average temperature of 80.2°, the cement 100°, the river water 74.4°, the cooled mixing water 42.4°, and the concrete placing temperature 72.2°. It will be noted that for June, 1939, for a five-day period when the water was not artificially cooled for the same shift, the average temperatures were: Air 83°, cement 107°, river water 75°, and the concrete casting temperature 81°. Thus it will be noted that the casting temperature of the concrete for June, 1939, was reduced about 9°. In the paper it was stated (see heading "Washing, Rinsing, and Cooling the Aggregate") that "The effective reduction in concrete placing temperature by refrigerating the mixing water was about 6°." Of course, it must be admitted that, if a temperature of 32° had been maintained for the pre-cooled water, the casting temperature would have been reduced another 2°, which would have been justifiable. The pre-cooling of the mixing water at Hiwassee Dam was an afterthought and was not effective until September, 1938. An old refrigerating plant that was available was installed so the minimum temperature for the mixing water was set at 35° or 36° to prevent the freezing of the pipes. In the case of a slow placing schedule, something less than 1 ft per day as was adopted for Hiwassee Dam, the advantage gained by pre-cooling may be lost to a limited extent.

The writer agrees with Mr. McHenry in regard to cooling the concrete in place artificially. He advocated the placement of cooling pipes and the circulation of cool water through the concrete of the dam before construction was begun, but this plan was not adopted by those in higher authority. The embedment of cooling pipes in local regions to reduce cracking, maintain uniform volume, and for other purposes resulted, and was fully justified by the results obtained.

The comparison of costs, as made by Mr. McHenry, between complete artificial cooling of the concrete in the dam and the combined cost of thin casting lifts, low casting temperatures, cooling the aggregate, and local cooling, is interesting. The estimates made by the writer for plant and operation and for embedding cooling pipes were about 17¢ to 18¢ per cu yd, which is higher than the cost of 14.4¢ at Grand Coulee Dam, primarily on account of the smaller yardage and the remoteness of the work. Pre-cooling the mixing water in conjunction with artificial cooling of the concrete in place would be much more effective because the low casting temperature would reduce the temperature during the early setting stage, and the circulation of cool water would prevent the dissipation of this pre-cooling during the long exposure period.

The discussion by Mr. Thomson is of great interest and of especial value concerning: The increasing care and concern to those in charge of concrete dams; the heavy maintenance and repairs of poorly constructed dams; the distribution of cement in the dam and its relation to heat, contraction, and expansion; the destructive effect of water and exposure; the danger of excessive use of water; the use of thin casting lifts; the use of diagonal keyways; and the writer's interpretation of the tables submitted with the paper.

To answer the inquiry of Mr. Thomson concerning the use of 10-ft lifts with complete artificial cooling, several assumptions must be made. If the costs of thin casting lifts, low casting temperatures, cooling the aggregate, and artificially cooling the concrete in place are eliminated in the tabulation of the cost of the crack prevention program, and the other four items retained, then savings could be made in addition to the cleanup of the pours. If the cleanup should be made at the same cost as at Hiwassee Dam (6¢ per sq ft) and the cost of circulating cooling water is placed at 17¢ per cu yd (as estimated in this discussion), then the cost of the crack prevention program using 10-ft lifts would be about 21¢ per cu yd compared with 26.5¢ at Hiwassee Dam. In the judgment of the writer the use of 10-ft lifts with the foregoing program would not be as effective for crack prevention as that used at Hiwassee Dam; and in addition other factors should be considered also.

Mr. Thomson refers to tests made to determine the effect of grinding in the mixers, and desires to know the purpose of the tests. They were made to determine the grading of the aggregate in the concrete after the mixing, in order that the engineers could make the necessary changes and adjustments to the grading of the aggregate as it was fed into the mixers. It was discovered early that the graywacke rock was inclined to break down in the mixers and produce an excess of fine material. The grinding of the aggregate in the mixers, including the tests made and conclusions drawn, is important and could easily be made the subject of a paper, so time and space will not permit complete discussion in great detail. The writer feels, however, that a brief résumé may be of interest. The rock and sand seemed to break down to a minimum size, between 200-mesh and 325-mesh, but there was practically no aggregate of a smaller size. During the summer of 1938 a large number of small "grab" samples (from 100 to 125 lb) were taken from the concrete at the mixer discharge, which were wet-screened to give an average grading from the mixer, and were compared with the average theoretical grading going into the mixer. The tests indicated a large increase in the − 100-mesh material, so this size was reduced from 16% to 12% in the finished sand by wet classifying. These small tests showed increases in the − 100 fines from 100% to 200%. The tests also showed an increase of the material between 28-mesh and 100-mesh during the mixing, which suggested a reduction of the quantity of sand. It may be noted in passing that no precedent was available as a guide to determine the most desirable grading or to make adjustments to compensate for the grinding. It was found that other aggregate material, which was believed to be stable, showed considerable change due to grinding, but not perhaps to the same extent as the graywacke.

In order to obtain more reliable data than those from the small "grab" samples, it was decided to analyze some full 2½-cu-yd batches. In the test batches it was necessary to omit the cement because the time for wet-screening a batch of that size was about a week for several men. In place of the cement, − 100-mesh classifier fines were substituted by absolute volume in order to simulate as nearly as possible the mixing action when cement was used. Six 2½-cu-yd batches were analyzed, three of which were charged and mixed for 2½ min in the regular manner and three of which were charged on what was

designated a "split batch" charging, and mixed for a total of $2\frac{1}{2}$ min. The "split batch" cycle was charged in two parts: First, the water, cement (or − 100-mesh fine substitution), sand, and two smaller sizes of rock were charged into the mixer and mixed for $1\frac{1}{4}$ min; and second, the coarse rock and cobbles were charged, and mixing continued for another $1\frac{1}{4}$ min. The objective, of course, was to reduce the amount of grinding, since the grinding, it was assumed, was largely a result of "ball mill" action with the coarse aggregate grinding the sand. This method of charging and mixing was adopted for "face" concrete after July 17, 1939. Changes in grading of the coarse aggregate—particularly cobbles, coarse rock, and medium rock—were small. The amount of material larger than 4-mesh changed only slightly after mixing, and a comparison of these amounts (see Table 10) indicates that very little material is

TABLE 10.—Comparison of Aggregate Grading for Mass Concrete Before and After Mixing[a]

(Expressed as a Percentage Change Based on Volume of Material Before Mixing)

Test No.	Standard Screen Size						
% passing: % retained:	.... 4-mesh	4 8	8 14	14 28	28 48	48 100	100 Pan
			(a) Regular Charging				
6	−1.0	−26.8	−11.5	−12.2	0	+41.9	+78.1
2	−3.0	− 6.8	−12.2	− 7.9	+41.2	+13.9	+47.2
1	−5.0	−20.0	−12.5	− 7.7	+27.6	+61.9	+58.2
			(b) Split Charging				
3	+2.0	−31.0	−11.5	− 7.3	+11.1	+23.7	+22.6
4	0	−29.7	−10.4	−10.5	+19.5	+25.0	+25.0
5	0	−27.4	−12.2	−12.8	+10.8	+ 5.7	+67.7

[a] Minus (−) indicates that the quantity after mixing is less than it was before mixing; plus (+) indicates that the quantity after mixing is greater than it was before mixing.

lost from the coarse aggregate into the sand classification. The results for the full batch tests altered the conclusions reached from the small "grab" samples. The large tests showed an average increase of about 50% for the − 100 classification.

Mr. Thomson raises a question as to the cost of the dam. The approximate costs kept at the dam are $13,285,000, to which should be added $2,379,000, representing other general accounts, and $1,150,000 for land acquisition, making a total of $16,814,000. If it were possible to separate, physically, the power house, penstocks, and power equipment from the dam, the $13,285,000 could be broken down further into these items: Dam, $10,226,000; power house, substation, turbine, generator, and miscellaneous power-house equipment, $2,001,000; reservoir relocations, $765,000; and reservoir clearing, $295,000. Mr. Thomson fixed the cost of the dam at $12,000,000, whereas the dam and power house cost $12,226,000; therefore the expenditure of $210,621, or 1.72%, as a factor for crack prevention and longer life seems reasonable.

Mr. Waugh takes exception to the writer's claim for reduction in the cost of the crack prevention program by the saving in cement. He also deducts the cost of the scientific program, \$29,000; the cost of cooling the aggregate, \$3,059; the cost of curing and winter protection, \$14,075; and a part of the cost of steel reinforcement, \$11,053, from the estimated cost of \$210,621 for the entire program. The net result of the deductions by Mr. Waugh is to show an increased cost of \$30,434 for the program, whereas the writer showed a saving of \$34,407. These items will be taken up briefly in the foregoing order, which is the same as that used by Mr. Waugh.

In the paper, the writer used the phrase "predicted cement content of one barrel per cubic yard" advisedly and after careful consideration. In other words, it seemed that there was no hope of using less than 1.00 bbl of cement for the interior mass concrete and 1.25 bbl for the face, using modified cement (Type B) and graywacke rock and sand. During the early period of the Hiwassee project, before the type of cement or cement content had been decided upon, 1.05 bbl of cement per cubic yard for the mass concrete in the interior of the dam was generally considered, especially using sand manufactured from graywacke rock. Consideration was even given to the importation of quartz sand from Georgia because of the breaking down of the graywacke sand. Exhaustive tests were made in the laboratory to determine the suitability of the various types of available aggregate, with the result that it was determined to use both graywacke rock and sand. Engineers of the design department and others in authority advocated 1.05 bbl of cement for the interior mass concrete of the dam. At several conferences the writer advocated 1.0 bbl per cu yd of modified cement (same as Norris Dam) for the mass concrete, but it was not adopted.

It was only after the consultants (Messrs. Carlson and Davis) had recommended[8] low-heat cement and low cement content that any cement content less than 1.0 bbl per cu yd was considered (to the knowledge of the writer). In their report they suggested that the desired results would be most nearly obtained as follows:

(1) By using a low-heat cement;

(2) By using for the interior of the mass a concrete of low cement content, and for the exposed faces a concrete of normal cement content;

(3) By a controlled placing schedule, using thin lifts immediately above foundations and immediately above surfaces of concrete that had been placed for a considerable period; and

(4) By maintaining low casting temperatures.

In their report they further stated:

> "A trial mix containing 0.75 bbl. per cu. yd. has been shown to be very plastic and workable and to exhibit almost no water gain. * * * preliminary investigations would seem to indicate that a cement content of 0.75 bbl. per cu. yd. might be employed with a water-cement ratio of not greater than 0.85, * * *. It is recommended that for the mass of the

[8] "Problems Pertaining to the Concrete for Hiwassee Dam," Report by Roy W. Carlson and Raymond E. Davis, Cons. Engrs., to Carl A. Bock, Asst. Chf. Engr., April 11, 1938, pp. 2 and 7.

dam there be employed a concrete for which the water-cement ratio is about, but not more than, 0.85 and for which the cement content is not more than 0.8 nor less than 0.75 bbl. per cu. yd."

Table 11, from the official records of TVA, is submitted to show the cement content of the dams designed and constructed previous to the Hiwasse Dam.

TABLE 11.—CEMENT CONTENT OF TVA DAMS

Description	Pickwick	Wheeler	Guntersville	Norris	Chickamauga	Hiwassee
Volume (cubic yards)	626,919	612,857	286,312	968,119	489,169	783,154
Cement Content:						
Barrels	693,956	774,054	369,442	1,061,190	642,783	741,127
Barrels per cubic yard	1.105	1.260	1.290	1.096	1.310	0.946

The amount of saving by the reduction in cement used can be obtained by three methods: First, by the actual number of barrels of cement used for the 596,431 cu yd of interior mass concrete, compared with the predicted one barrel per cubic yard, which gives 114,499 bbl at $2.14, or $245,028; second, by applying the saving of 0.20 bbl per cu yd for the 596,431 cu yd, which gives 119,290 bbl of cement at $2.14, or $255,280; and third, by taking the total yardage of concrete at Hiwassee Dam of 794,439 cu yd and applying the difference between the average cement content at Hiwassee Dam and the average at Norris Dam (as shown in Table 11) at 0.15 bbl per cu yd, which gives 119,166 bbl at $2.14, or $255,015. After all, the amount of saving in cement is a matter of judgment. The writer believes the lowest sum ($245,028), which was used in the paper, to be fair. Mr. Waugh states that "At Norris Dam a cement content of 0.90 bbl per cu yd for the interior concrete was adopted." According to the record, that is correct, but the adopted content was not realized in practice. It is found from the record that, during the first two months of concreting at Norris Dam, 1.10 bbl was used; then the content was changed to 0.95 bbl and later adopted to 0.90 bbl. Evidently the actual average of placement was in excess of 1.00 bbl because the average of all mass concrete interior and face (for 871,763 cu yd) was about 1.061 bbl per cu yd and the average for 968,119 cu yd was 1.096 bbl.

The discussion by Mr. Waugh, of studies of temperature cracking in mass concrete, is valuable.

Mr. Waugh admits that some benefit was received during construction from the scientific program, and the writer admits that the benefit did not equal the total cost of $29,000, but the clearest explanation is made by Mr. McHenry when he states

> "The inclusion of the $29,000 item for the scientific program is likewise of interest; this inclusion acknowledges a debt to the similar research investigations of the past that are to a large extent responsible for the success and economy of the Hiwassee control program."

Mr. Waugh admits a small benefit from the cooling of the aggregate, but the charge (explained in detail in the paper) also is small. On the other hand, the reduction in casting temperature of the concrete amounted to about one degree

The design of the inspection tunnels (being flat on the top and bottom with chamfered corners and heavy horizontal steel reinforcement above the roof and below the invert), and the placement of interior mass concrete around the exterior of the tunnels, produced a job practically free from cracks; so the inclusion of the cost of the steel reinforcement in the crack prevention program seems reasonable. Only that part of \$74,600 (the cost of the curing and winter protection, which could be attributed to the slow setting tendency of low-heat cement), or \$14,075 as explained in the paper, was charged to the crack prevention program. All of the items of cost were obtained from the records of the cost engineer, and the final figures reflect his judgment, and that of several others familiar with the work.

The writer desires to express his appreciation to those participating in the discussion of this paper.

AMERICAN SOCIETY OF CIVIL ENGINEERS

Founded November 5, 1852

DISCUSSIONS

FOMULAS FOR THE TRANSPORTATION OF BED LOAD

Discussion

BY H. A. EINSTEIN, ASSOC. M. AM. SOC. C. E.

H. A. EINSTEIN,[32] ASSOC. M. AM. SOC. C. E. (by letter).[32a]—All discussions of this paper definitely show how little is known about bed-load movement. Mr. Shulits, for instance, asks of what avail is all this research if different investigators can derive different results (formulas) from the same basic data. This is the fate of all empirical research, because, when one tries to fit a mathematical curve through a group of scattering points, it is always possible to use fundamentally different functions that will all fit the points equally well within a certain range. Only by enlarging the range of measurements into a region where the different curves diverge can one choose among them. In this respect the writer was extremely fortunate in having the Swiss experiments[5] available, since they extend the range of experimental data considerably toward the greater depths and coarser grains.

In 1934 the writer developed[7] what is today generally known as the Meyer-Peter formula. This formula was based on the same experiments as the ϕ-ψ formula and satisfied a great number of them, especially those with coarse sediment. Accordingly, it proved very useful and dependable in its application to mountain streams with steep slopes and coarse sediment. It is well known that this formula will not describe the transportation of fine sediment, such as Gilbert's sands, smaller than 1 mm in diameter.

The ϕ-ψ method of describing bed-load transportation, and the Meyer-Peter formula, are both based on similar principles, but the ϕ-ψ method is more flexible. For this reason it can be correlated to all known experiments. The parameter ϕ is a measure of the intensity of transportation and seems to be of

NOTE.—This paper by H. A. Einstein, Assoc. M. Am. Soc. C. E., was published in March, 1941, *Proceedings*. Discussion on this paper has appeared in *Proceedings*, as follows: June, 1941, by Joe W. Johnson, Assoc. M. Am. Soc. C. E.; and September, 1941, by Messrs. A. A. Kalinske, O. G. Haywood, Jr., Samuel Shulits, and John S. McNown.

[32] Hydr. Engr., SCS, U. S. Dept. of Agriculture, Greenville, S. C.

[32a] Received by the Secretary October 29, 1941.

[5] "Der Geschiebetrieb als Wahrscheinlichkeitsproblem," von H. A. Einstein, *Mitteilung der Versuchsanstalt für Wasserbau an der Eidgenössische technische Hochschule in Zürich*, Verlag Rascher & Co., Zürich, 1937.

[7] "Neuere Versuchsresultate über den Geschiebetrieb," by E. Meyer-Peter, H. Favre, and A. Einstein, *Schweizerische Bauzeitung*, Vol. 103, No. 13, March, 1934.

special importance, as most known bed-load formulas are valid only in certain ranges of ϕ. However, even if this method permits all measurements to be represented, it is still doubtful if any one mathematical curve will describe the entire range of known experiments. Conditions in large rivers with fine sediment are characterized by still higher values of ϕ, for which no flume experiments have ever been made. Any final curve should certainly embrace this range of higher values.

The general pattern of research in the bed-load problem includes, since the first work of Gilbert[8] and Schoklitsch,[33] the following three steps: (1) Flume studies on beds of uniform grain; (2) extension of the results to mixtures; and (3) application to rivers under natural conditions. The ϕ-ψ method of representation is developed from the first stage alone. Its application to problems of the second step has been tried by the writer, using the experiments[10] made at Vicksburg; but he was forced to the conclusion that some kind of sorting must have occurred, contrary to Captain Haywood and the testimony of Fig. 4. Bed-load investigations conducted by the U. S. Soil Conservation Service in a small creek seem to give an explanation for this discrepancy. In these experiments the ϕ-ψ graph also shows points between curves (2) and (S) (Fig. 3(b)), and the composition of the bed as a whole is definitely the same as that of the bed load moving on it. In the top layers of the bed, however, a definite sorting took place, accumulating the coarse grains in a certain layer and the fine grains above and below. Depending on the extent to which this coarse layer is exposed, the rate of transportation may change for any one flow. Very likely the same thing has happened in the Vicksburg flume, and therefore could not be detected by samples as represented in Fig. 4. Investigations by the Soil Conservation Service on this problem are still in progress (November, 1941) and may soon yield definite results. If the writer's explanation is correct, the entire space between curves (2) and (S), Fig. 3(b), would constitute a possible range for non-silting and non-scouring channels. Elaboration on the extreme importance of this question is probably not necessary.

The average single step as introduced in statement (c) of the "Introduction" has been determined in a great number of measured distributions similar to the one presented by Mr. McNown in Fig. 8(b). Their averages, not the steps themselves, are independent of the flow, but seem to vary considerably with form and roundness of the particles. Further discussion of this question does not seem desirable at this time, as the complicated formulas were presented in 1937,[5] and relatively few readers would be interested in their derivation.

Professor Kalinske asks what particles are included in the description of bed load given in the "Introduction." This is very easy to answer: All particles in any flume experiment with uniform material, rolling or in saltation. It just happened that under the conditions of all these experiments particles did not seem to go into suspension. There is no single flume study with uniform

[8] "The Transportation of Debris by Running Water," by Grove Karl Gilbert, *Professional Paper No. 86*, U. S. Geological Survey, Washington, D. C., 1914.

[33] "Ueber Schleppkraft und Geschiebebewegung," by A. Schoklitsch, Verlag Wilhelm, Engelmann, Leipzig & Berlin, 1914.

[10] "Studies of River Bed Materials and Their Movement, with Special Reference to the Lower Mississippi River," *Paper No. 17*, U. S. Waterways Experiment Station, Vicksburg, Miss., January, 1935.

grain available in which bed load and suspended load have been measured simultaneously.

The term "critical tractive force" is so familiar to every one that it seems almost impossible to do without it. Unfortunately, it is a condition that does not exist in nature. If it did, the points somewhere near the upper end of curve (2) would approach a horizontal line. The top points, representing a rate of transportation of some few pounds per hour in a 6-ft wide flume, would definitely be called zero transportation by all observers. If this small rate is measured, however, the data are found to fit curve (2). The writer would readily change his opinion about this point if any one could cite some experiments that show this break in curve (2); but apparently the break does not seem to exist.

It is clear that for the practical engineer the concept of a critical tractive force or critical value of ψ is very useful, since it indicates the flow under which a natural or artificial bed will be stable. For this purpose one must remember that an exact limit of transportation does not exist, but that the erosion can be reduced to any safe amount by choosing the proper ψ-value according to the expected duration of the critical flow.

The final important point under discussion is the influence of turbulence. R. A. Bagnold[34] shows indirectly that turbulence is greatly responsible for the transportation of bed load. Describing his transportation experiments conducted in a special wind tunnel, he states:

> "Sand placed at the mouth itself was never disturbed even at the highest speeds used, despite the fact that the drag and the normal velocity gradient must be a maximum there."

He does not give any explanation for this fact, but it is obvious that at the mouth of the wind tunnel turbulence has not developed yet and that this lack of turbulence can be the only reason for the failure of the sand to move.

The term turbulence includes all pulsatory velocities in the flow. The influence of vertical turbulence velocities on a moving grain is the action typical for suspension and has been excluded from the problem as such. Such vertical velocities might to some extent be responsible for the deviation of the finest grain sizes in Fig. 3(*a*) by increasing λ. Any influence of vertical velocities on grains in the bed, however, is impossible, as those vertical turbulence velocities do not exist in the immediate proximity of the bed (the water would have to stream out of the bed). Therefore, only horizontal velocities are available to move particles out of the bed. Since Mr. Bagnold proved that the average velocity without turbulence is definitely not able to cause the movement, it must be concluded that the horizontal turbulence or pulsations are responsible for the movement. (Mr. Kalinske may remember that in Mr. Shields' experiments the flow outside the laminar sub-layer was definitely turbulent and that the disturbances due to the eddies reached well into this sub-layer.)

---

[34] "The Movement of Desert Sand," by R. A. Bagnold, *Proceedings*, Royal Soc. of London, Series A, Vol. 157, 1936, p. 600.

Mr. Rubey[4] chose as the title of his paper: "The Force Required to Move Particles on a Stream Bed," thus indicating that his investigations deal with the forces acting on particles, like particle *a* in Fig. 9, that are on the bed rather than on particles like *b* which are in the bed. Close observation of bed-load movement shows that most moving particles are dislodged from positions in the bed. Thus, it is easily understood why Captain Haywood found a constant $A_4 = 0.01$ for the lift on the particles, and this is also the reason why the writer always refers to a "lifting" force. The low value of A_4 in itself excludes the possibility of any impact or push, such as would take place in the case of particles in position *a*, Fig. 9. Captain Haywood's rather high value of $t = 0.32$ sec may be correct. An explanation could be that before removal the particles first must be freed from the surrounding particles by small vibrating movements. In motion pictures of bed-load movements these vibratory movements can be seen readily.

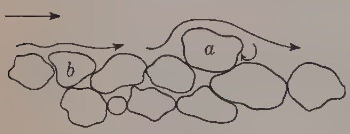


FIG. 9

The writer believes that he has answered the most important questions raised in discussion. Much could be added. In conclusion he would like to emphasize the fact that the problem is far from being solved and that a great number of the most important questions pertaining to sediment transportation cannot be answered because the necessary measurements do not exist or because they are incomplete. Every one investigating problems of sediment transportation would do well to keep Mr. Johnson's remarks in mind.

[4] "The Force Required to Move Particles on a Stream Bed," by William W. Rubey, *Professional Paper No. 189-E*, U. S. Geological Survey, Washington, D. C., 1938.

AMERICAN SOCIETY OF CIVIL ENGINEERS

Founded November 5, 1852

DISCUSSIONS

DESIGN OF ACCELERATION AND DECELERATION LANES

Discussion

BY ADOLPHUS MITCHELL, ASSOC. M. AM. SOC. C. E.

ADOLPHUS MITCHELL,[21] ASSOC. M. AM. SOC. C. E. (by letter).[21a]—In preparing the paper on acceleration and deceleration lanes it was recognized that the available data on which the limit of motor deceleration and other constants were based was not sufficient to allow the selection of final values. For this reason, it was hoped that those discussing the paper would do so in a constructive manner and present more conclusive evidence for the determination of these factors. It is gratifying that practically all discussions were kept on this high plane.

In summarizing the discussions submitted, an effort will be made to isolate the controversial elements and to modify the recommendations of the paper where it appears to be justified.

Deceleration Lanes.—The weaving distance was given particular attention by Messrs. Loutzenheiser and Moyer and Davidson. It is gratifying to note that they all agreed with the writer that his values were acceptable as minimums. Messrs. Moyer and Davidson appear to have gone further into the problem than any one else and in a very scientific manner. They state that "By holding the side friction to a maximum value of $f = 0.16$ recommended by the author, the minimum lengths in the weaving distance obtained in these tests checked rather closely with the values given in Fig. 2." However, on the basis of a spiral shown in Fig. 15, they have arrived at a length somewhat in excess of those given in Fig. 2, and this bears some comment.

In stating that "The assumption of an S-path of travel is a theoretical expedient entirely," and that "there is no doubt that drivers would find a spiral path more natural than a reverse circular path," the writer did not have reference to the spiral employed by Messrs. Moyer and Davidson. This spiral

NOTE.—This paper by Adolphus Mitchell, Assoc. M. Am. Soc. C. E., was published in March, 1941, *Proceedings*. Discussion on this paper has appeared in *Proceedings*, as follows: May, 1941, by Messrs. H. F. Holley, D. W. Loutzenheiser, Hawley S. Simpson, and Milton Harris; June, 1941, by T. F. Hickerson, M. Am. Soc. C. E.; and September, 1941, by Messrs. W. L. Waters, R. A. Moyer and Donald T. Davidson, and Stephen E. Butterfield.

[21] Senior Traffic Engr., State Highway and Public Works Comm., Raleigh, N. C.

[21a] Received by the Secretary October 29, 1941.

was developed by railroad engineers on the basis of providing for a transition from a tangent to a circular curve in a manner that would increase the centrifugal force acting on the vehicle uniformly from zero at the end of the tangent to that determined by the circular curve at the end of the transition. This would permit the elimination of all "pants friction" provided that the superelevation was increased uniformly from zero at the end of the tangent to that of the circular curve at the end of the transition, on the condition that a constant speed was maintained. In the present problem it will be impracticable to superelevate the pavement in this manner and to assume that the speed will not vary throughout the weaving distance. For these reasons, Messrs. Moyer and Davidson not only failed to present a formula for the "natural path" but also failed to present one resting on a sound theoretical basis. Although the discussion of the weaving distance gives the impression of providing a basis for longer minimum lengths, examination of the throw (p) of the spiral shows this to be purely theoretical (amounts to p = 0.35 ft in the problem of Fig. 7, although L is equal to 505 ft, using f = 0.16, W = 17, V = 50, and C = 3) and not at all commensurate with the accuracy with which automobiles are steered. Their comparison of Fig. 16 with Fig. 2 is not valid. All curves in Fig. 2, except one, are based on a width of lane varying with the speed as listed under the heading "Deceleration Lanes." The values of Fig. 2 are nearer 70% less than those of Fig. 16 than the 40% listed by Messrs. Moyer and Davidson.

Their remarks regarding C in Eq. 18 were not clear since C is a constant, the evaluation of which is not final. They themselves recommend using C as a constant for any given design speed.

Mr. Simpson suggests that motor deceleration be assumed equal to 40 miles per hr, and Messrs. Moyer and Davidson suggest 40 or 50 miles per hr. The writer suggested that "40 miles per hr for a 70-mile-per-hr highway and 30 miles per hr for a 50-mile-per-hr highway" be assumed. Since nearly all of those taking part in the discussion agreed that deceleration lanes determined on this basis are too long, it is suggested that the assumption of motor deceleration to 50 miles per hr be used for a 70-mile-per-hr highway and 40 miles per hr for a 50-mile-per-hr highway.

Most of those discussing the braking distance agreed that a somewhat higher coefficient of friction should be adopted than those shown in Fig. 5. Messrs. Moyer and Davidson suggested a value of f = 0.25, on the basis of actual tests. The writer is inclined to agree with them and suggests that this value be used. This would result in the following equation:

$$s = \frac{\bar{V}^2 - V^2}{7.5 \pm 0.3\,G} \qquad (25)$$

There was very little comment on funneling or providing a decreasing width of lane as speed decreases. Apparently no one knows anything about this subject at the present time.

Very little was written about the insulation strip for the deceleration lane. The writer is inclined to shorten this strip to a length determined by the emergency stopping distance so as to provide a more open throat than that

theoretically required. The insulation strip is necessary to insure correct use of the deceleration lane, as has been proved by experience. Proper design will eliminate the hazard of introducing an island point.

The changes agreed upon would result in a length of deceleration lane of 465 ft instead of 650 ft for the problem of Fig. 7.

Acceleration Lanes.—It appears that there is no definite agreement of the distance required for full acceleration. Mr. Loutzenheiser does not think the writer was justified in extending the data for level ground to include the effect of gradients. It has been the experience of Ernest E. Wilson, director of the General Motors Proving Ground, that "The hill climbing abilities of a passenger car can be very closely determined from theoretical equations if acceleration data taken on a level road are used in the calculations."

The insulation strip along the acceleration lane was an item of considerable interest. Mr. Loutzenheiser would either shorten it or omit it altogether. Mr. Butterfield thinks it is all right as presented. Messrs. Moyer and Davidson suggest that it be shortened to one half of the length of the acceleration lane. The writer suggests that it be shortened to the point where it is possible for entering traffic to attain the average speed of the adjacent highway.

There was considerable diversity of opinion as to the braking section provided at the end of the lane. Mr. Harris does not think it is necessary, and Messrs. Moyer and Davidson would use a low-cost paved surface. Since this section amounts to no more than improved shoulder, the writer is inclined to agree with Messrs. Moyer and Davidson.

Messrs. Simpson and Moyer and Davidson would construct deceleration lanes only, whereas the others appeared to think that the acceleration lane had some merit, particularly Messrs. Harris and Butterfield. After studying the various comments on the subject, the writer still fails to understand why acceleration lanes are not important for dual-lane highways. Where traffic is heavy, it is even more important than where it is light, because it would be necessary to use signals to enable traffic to enter. The only problem seems to be that of persuading traffic to move from the outside to the inside lane in order to allow traffic to enter from the acceleration lane (signs, signals, markings, and devices are available for this purpose). Since it is relatively easy to enter a traffic stream when approximating the speed of the automobiles thereon, it would require a smaller distance between vehicles on the highway in order to enter it than would be the case when an automobile had to enter at a very low speed as would be the case at a T-intersection. The operation of "stop" lights would introduce delays on the highway that engineers have come to recognize as equivalent to a monetary loss. The cost of such delays should be estimated to determine if they would justify the construction of the acceleration lane.

Messrs. Loutzenheiser, Harris, and Waters mentioned the need for additional data on acceleration and deceleration lanes, some of which were furnished by Messrs. Moyer and Davidson. Data are needed on the form of the "natural path," the maximum permissible differential in the speed of cars on the highway with that of merging traffic, and other factors involved in the design of acceleration and deceleration lanes.

Conclusion.—Mr. Holley's example of the Fair Oaks Avenue crossing of Arroyo Seco Parkway was most interesting; however, its value was greatly reduced by his failure to give speeds, grades, and dimensions. It is significant that he states, "Mr. Mitchell's paper provides an objective toward which designing engineers should strive."

Mr. Loutzenheiser appears to believe that the writer's paper is "an academic discussion, somewhat extended in detail, of a design approach made in a paper[9] published elsewhere, in 1940." The writer's paper was written in 1939 and submitted to Yale University, at New Haven, Conn., in 1940, through the Yale Bureau for Street Traffic Research, in partial fulfilment of the requirements for a certificate in street traffic control. There is a marked lack of similarity between the two treatments.

The writer is indebted to Professor Hickerson for checking the development of the equations used.

[9] "A Policy on Intersections at Grade," Am. Assoc. of State Highway Officials, 1940.

AMERICAN SOCIETY OF CIVIL ENGINEERS

Founded November 5, 1852

DISCUSSIONS

SURFACE RUNOFF DETERMINATION FROM RAINFALL WITHOUT USING COEFFICIENTS

Discussion

By W. I. Hicks, Esq.

W. I. Hicks,[35] Esq.[35a]—A comprehensive outline of a method of computation, based on the analysis and synthesis of the factors governing runoff from a city block, is presented in this paper, which points the way to an extension of the computation to a complete drainage system. On account of the wide scope of the subject and the limitations of space, the text must be read carefully to enable one to grasp the entire subject matter.

The writer has been investigating rainfall-runoff phenomena in Los Angeles, Calif., for a number of years, with the result that a schedule of procedure identical with the six stages listed by the authors (see heading "Application to Hydraulic Engineering") was evolved:

I—A precipitation pattern between "intermediate" and "delayed" was selected;

II—The infiltration curve was based on small test plots and the analysis of rainfall-runoff records gathered from local drainage areas (see Fig. 16);

III—Antecedent precipitation was determined by probability studies and used to weight the basic infiltration curves;

IV—Net rainfall curves were derived for different classes of development after consideration of—

V—Infiltration curves, depression storage varying from 0.10 in. for heavy soils to 0.20 in. for sand; and

VI—The net rainfall curves were translated into hydrographs after consideration of the various forms of detention.

The summation-of-hydrograph method of computation is well suited to Los Angeles terrain where the intensity of a storm of given frequency varies

Note.—This paper by W. W. Horner, M. Am. Soc. C. E., and S. W. Jens, Assoc. M. Am. Soc. C. E., was published in April, 1941, *Proceedings*. Discussion on this paper has appeared in *Proceedings*, as follows: June, 1941, by L. L. Harrold, Assoc. M. Am. Soc. C. E.; and October, 1941, by Messrs. C. E. Ramser, LeRoy K. Sherman, A. J. Schafmayer, C. S. Jarvis, G. W. Musgrave, and F. L. Flynt.

[35] Asst. Civ. Engr., Storm Drain Div., Bureau of Eng., Los Angeles, Calif.

[35a] Received by the Secretary August 8, 1941.

as much as 2 to 1 in a single drainage area; the reason for the variation is the mountainous character of a part of the area.

Apart from the agreement in the general principles, there were some variations and differences on various details of more or less importance.

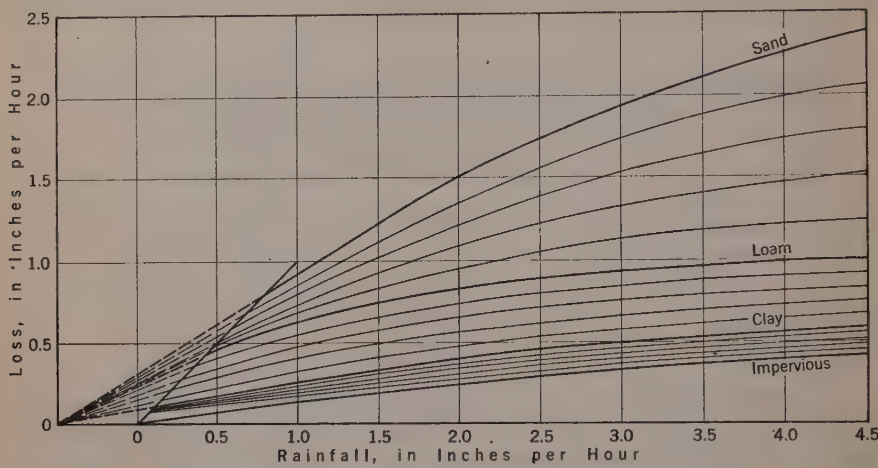


FIG. 16.—RAINFALL RATE—LOSS RATE CURVES

In delineating the precipitation curve, the mass curve was used in preference to the intensity diagram because it was believed that the volume of runoff was of major importance. Likewise, the influence of detention was introduced by suspending the varying depth of storage under the net rainfall curve; and the time functions of the hydrograph were approximated by analyses of hydrographs from drainage areas of various sizes and stages of development.

The general shape of the authors' infiltration-capacity curve is in agreement with local data. Analyses of rainfall-runoff records for a four-day storm culminating on March 2, 1938, show a downward march in capacity to practically zero (the zero capacity was a combination of low infiltration and seepage flow). Because the local rainy season comes in the winter when evaporation is low, because of the delayed position of the peak rainfall, and because of relatively high antecedent precipitation, the flat part of the curve is used, whereas the authors use the sharply descending part of the curve for the "thunderstorm" which breaks a hot, dry period.

The curves in Fig. 16 differ from the authors' assumption of comparative independence of rainfall intensity under turf cover in that the higher intensities of rainfall produce a more complete coverage of the imperfect plane surface of urban yards and so increase the infiltration capacity.

In the authors' typical block, they assume that drainage from impervious surfaces does not flow over turf on its way to the gutter or alley. Analysis of the various classes of urban residential districts indicates that walks, driveway strips, and small porches usually discharge on to turf and that many roof downspouts discharge into flower beds; the total of these areas average from 15% to 20% of the entire area. If this assumption is granted, the gross

rainfall for the pervious area is increased by the runoff from these areas, and the impervious supply volume is diminished correspondingly.

In the matter of overland flow and surface detention, limited experimental data were taken locally. The results of the experiments are included for the purpose of discussing the paper and to make them available for use.

The apparatus consisted of:

(1) A rectangular wooden trough 100 ft long by 0.96 ft wide mounted on a series of wooden standards and capable of being adjusted to different slopes.

(2) A 2-in. galvanized pipe, capped at both ends and tapped at intervals to receive nipples, mounted in the wooden standards above the trough. Below this pipe and connected to the nipples were suspended brass sprinkler pipes with close-spaced sprinkling orifices.

(3) A hose connection made from the 2-in. pipe to the valve of a 100-gal tank on a platform set uphill from the apparatus.

The types of surfaces tested were:

(*a*) Tar and sand corresponding to tar-paper roofing, sidewalk, concrete paving, and similar surfaces;

(*b*) Tar and gravel corresponding to the roofing of that type and rough-finish asphaltic concrete paving; and

(*c*) Clipped sod selected as being representative of the average city lawn. Tar-paper baffles were set flush with the ground surface to prevent subsurface flow.

Slopes used varied from 0 to 7%. Lengths of flow were 100, 50, 25, and 10 ft. (The 10-ft runs were on a 10-ft by 10-ft adjustable platform.) Uniform rates of simulated rainfall varied from 0.5 to 7.0 in. per hr.

Data from these tests have been filed in the Engineering Societies Library.[36] They are quite erratic for the tar and sand surface on account of inability to maintain a true plane for so smooth a surface; the data for the other surfaces are more rational but require judgment for interpreting. The data were plotted on logarithmic paper and the following exponential formulas were derived:

Tar and Sand Surface.—

$$\delta_a = \frac{0.0136\, l^{0.318}\, \sigma^{0.535}}{S_0^{0.4615}} \qquad (17a)$$

and

$$t_c = \frac{1.3\, l^{0.323}}{S_0^{0.448}\, \sigma^{0.64}} \qquad (17b)$$

Tar and Gravel Surface.—

$$\delta_a = \frac{0.0257\, l^{0.384}\, \sigma^{0.351}}{S_0^{0.367}} \qquad (18a)$$

and

$$t_c = \frac{2.23\, l^{0.373}}{S_0^{0.366}\, \sigma^{0.684}} \qquad (18b)$$

---

[36] 29 West 39th Street, New York, N. Y.

Clipped Sod Surface.—

$$\delta_a = \frac{0.078\, l^{0.322}\, \sigma^{0.325}}{S^{0.281}_0} \quad \ldots\ldots\ldots\ldots (19a)$$

and

$$t_c = \frac{9.34\, l^{0.298}}{S^{0.302}_0\, \sigma^{0.785}} \quad \ldots\ldots\ldots\ldots (19b)$$

Except that S_0 denotes slope, as percentages in Eqs. 17, 18, and 19 (and in Eqs. 20a and 20b), the symbols used are those of the paper, t_c being measured from the beginning of the supply rate to the point of full runoff and S_0 = slope expressed as a percentage. It is to be noted that not the marginal but the

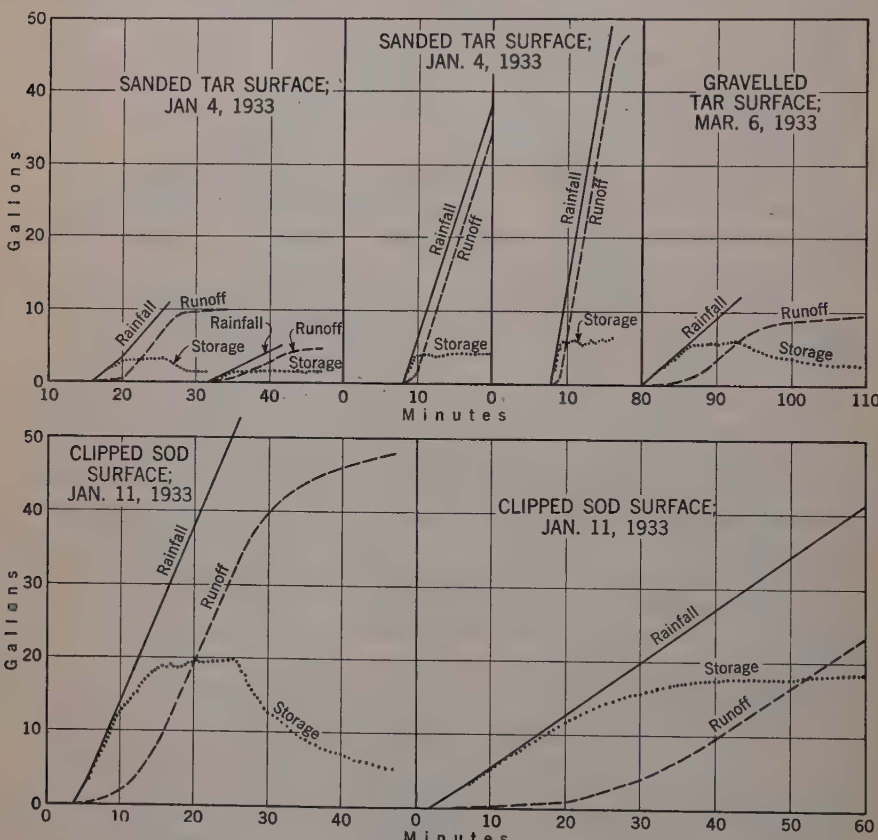


FIG. 17.—SAMPLE RUNS (TROUGHS, 100 FT BY 1 FT; AND S_0 = 0.03)

average depth is used. Nomographs were designed for the convenient solution of Eqs. 17 to 19. Sample runs are shown in Fig. 17.

From the experimental data, the curve in Fig. 18 was derived with percentage of time of concentration as abscissas and percentage of δ_a or storage as ordinates. It was based on data from a variable-flow trough, a clipped

sod plane surface, a tar and gravel plane surface, and a tar and sand plane surface. With these formulas and their corresponding charts, the alteration of the hydrograph of supply after flow over a plane surface can be computed as outlined in the paper (between Eqs. 4 and 5) except that a mass curve of supply and varying depths of storage are used instead of a supply-intensity curve and the variation of intensity of runoff from the plane surface.

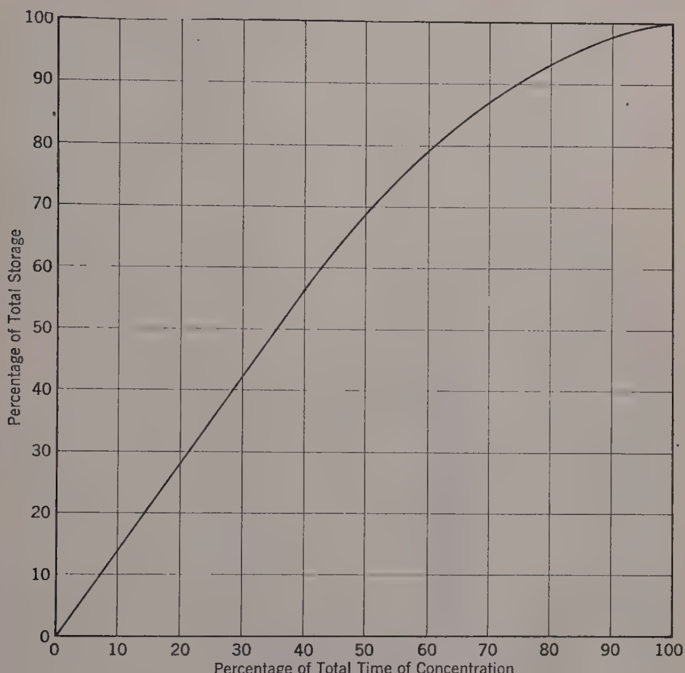


FIG. 18.—RELATION OF TIME OF CONCENTRATION TO STORAGE BUILD-UP

Results obtained by the two methods of treatment are consistent for all practical purposes. When one considers the unavoidable variations in roughness of surface of roofs, pavement, and turf, and the necessary assumption of average conditions, precise mathematical treatment is not to be expected.

In determining the effect of gutter storage on the hydrograph, the writer has followed the general principles stated by the authors. Two formulas similar to those for plane surfaces were computed on the basis of a 30-ft roadway ($n = 0.012$) with a 4.5-in. crown and no crossfall, which was selected as representing average conditions. A series of tests was performed on a V-shaped trough with equal increments of flow introduced at regular intervals along its length; the purpose was to test the manner in which storage was built up. The run on a flat grade supported the authors' remarks on the difference between the water-surface slope and the construction grade; the upstream

water slope was much flatter. The formulas are as follows:

$$\delta_a = \frac{0.00044\, l^{0.697}\, \sigma^{0.794}}{S^{0.417}{}_0} \qquad \text{(20a)}$$

and

$$t = \frac{0.0443\, l^{0.732}}{S^{0.39}{}_0\, \sigma^{0.252}} \qquad \text{(20b)}$$

in which t is time of flow with fully developed gutter storage; for t_c as in the other formulas multiply by 1.33 ±. Similarly to Eq. 19, a nomograph was designed for the solution of these formulas.

The hydrograph of runoff from the lot is transposed to the hydrograph of inflow to gutter inlet by varying the gutter storage in the manner outlined by the authors in the text following Eqs. 4.

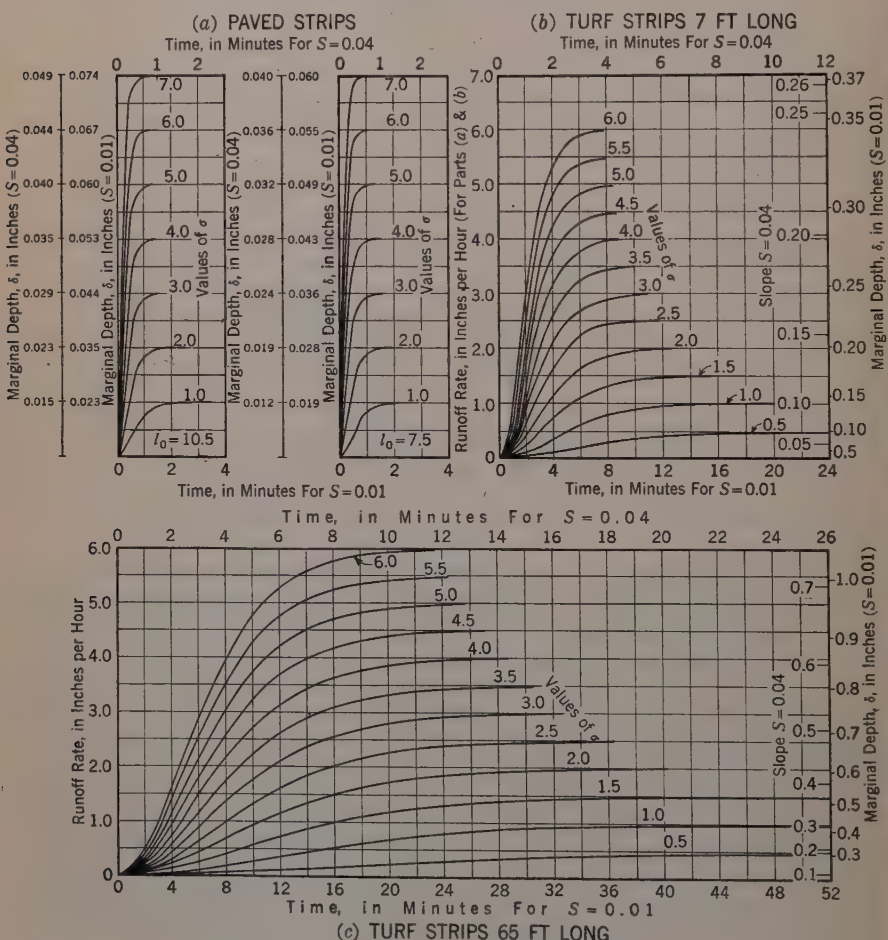


FIG. 9.—RUNOFF RATES, OVERLAND FLOW

Under "Summary and Conclusions: Conventional Urban Areas," the authors suggest, in general terms, the extension of the computation beyond the hydrograph for a single block; paragraph (*e*) indicates a process of summing such hydrographs at all points of inflow (see also text following Table 4). If this interpretation is correct, the volume of routine design work will be considerable and the chance of accumulative error will enter into the computations. A comprehensive experimental study of the hydraulics of the flood wave, confluence of hydrographs, and analytical comparisons of the hydrograph of the block with those of drainage areas of different sizes may yield information as to time factors and other elements of the composite hydrograph by which these objections can be avoided.

Acknowledgment.—The studies reported herein were made under the direction of L. W. Armstrong, M. Am. Soc. C. E., the field activity being supervised by H. W. Fraim.

Corrections for *Transactions:* In Eq. 3*b* change "$l^{0.5}\,\sigma^{0.25}$" to "$l^{0.5}\,\sigma^{0.5}$"; for Fig. 9 substitute the corrected Fig. 9 shown on page 1930. after Eq. 9*b*, insert: "With Q expressed in cubic feet per second, Eq. 9 becomes

$$k = \frac{60\,l}{7}\left(\frac{1}{936\,S^{0.5}}\right)^{0.75} \qquad (9c)$$

which expresses the value of k used in Eqs. 12"; and in the sentence that intro-

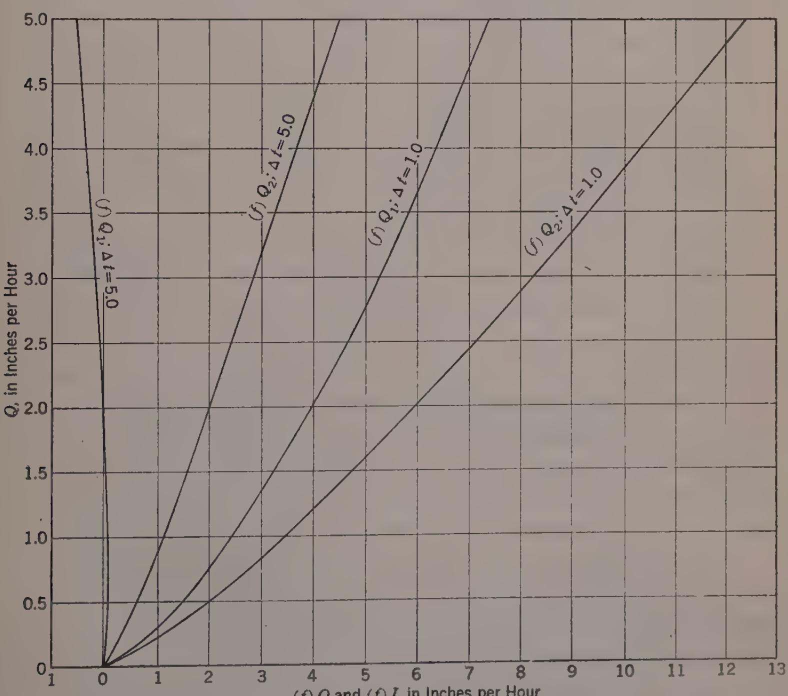


FIG. 13.—GRAPHS FOR THE SOLUTION OF THE STORAGE EQUATION FOR THE ALLEY

duces Eq. 10, change "with inflow I in inches per hour," to "with inflow I in cubic feet per second."

The curves of Fig. 13 are incorrect due to the erroneous inclusion of the factor "A" twice in Eq. 12*b*, once as shown in the numerator and again in the factor "k." Eq. 12*b* is correct if the value of "k" as given in Eq. 9*c* is used. The corrected Fig. 13, to be published in *Transactions*, is as shown on page 1931. Fortunately the effect of applying these corrected storage curves would result in only minor changes in the outflow hydrographs of Figs. 5, 6, and 7. The maximum increase in peak values being approximately 8%, there are relatively small changes in the peak flow rates originally published in Table 5. These corrections due to the incorrect curves of Fig. 13 do not warrant any changes in the other curves, the tables, or the text as originally published, nor in any of the discussion to date. See also corrections in June, 1941, *Proceedings*, page 1182.

AMERICAN SOCIETY OF CIVIL ENGINEERS

Founded November 5, 1852

DISCUSSIONS

EVALUATION OF FLOOD LOSSES AND BENEFITS

Discussion

By Malcolm Elliott, M. Am. Soc. C. E.

Malcolm Elliott,[16] M. Am. Soc. C. E.[16a]—The author correctly stresses the difficulties attending the use of depreciated property values in the evaluation of flood losses. In first place among these difficulties is the uncertainty as to the rate of interest to use in computing the capitalized value of the annual flood losses. However, lest it be understood that this difficulty is avoided by computing losses on an annual basis, the writer desires to emphasize that, regardless of whether the capital-loss or annual-loss methods are used, the determination of economic desirability of any flood-protection project depends on the adoption of a suitable rate of interest. The question raised by the author—"What rate?"—will inevitably plague the analyst in any case.

The difficulty can be illustrated by an assumed example pertaining to a flood-menaced area of 10,000 acres of corn land:

Benefits:		
Average flood losses, per acre per annum		$ 5
Average annual flood loss (10,000 × 5)		50,000
Costs:		
Cost of protection works		600,000
Annual interest and amortization combined, say 5%	$30,000	
Annual maintenance, operation, and all other annual charges	10,000	
Total annual charges		$ 40,000

$$\text{Ratio, } \frac{\text{Benefits}}{\text{Costs}} = \frac{50{,}000}{40{,}000} = 1.25$$

Note.—This paper by Edgar E. Foster, Assoc. M. Am. Soc. C. E., was published in May, 1941, *Proceedings*. Discussion on this paper has appeared in *Proceedings*, as follows: September, 1941, by Messrs. E. L. Chandler, E. F. Chandler, and Charles B. Burdick; and October, 1941, by Messrs. H. K. Barrows, Roger E. Amidon, Hyman J. Fine, and Otto F. Buzhardt.

[16] Col., Corps of Engrs., U. S. Army; Div. Engr., Upper Mississippi Val. Div., St. Louis, Mo.

[16a] Received by the Secretary November 7, 1941.

Can one conclude from this favorable ratio that the project is economically desirable? The capital investment in this instance may be considered the sum of the initial cost, \$600,000, and the capitalized cost of future maintenance and operation, say \$200,000, or a total of \$800,000. This is the equivalent of \$80 per acre and if, as often may be the case, other lands, flood-free and of equal fertility, may be bought in the vicinity for from \$75 to \$100 per acre, an investment of \$80 per acre for flood protection of the bottom lands would appear uneconomical.

The foregoing case is purely imaginary, yet there have been actual cases studied and reported in which, although the comparison of annual benefits and costs appears to justify the project, the capital investment seems out of proportion to the capitalized benefits. Why (as in this case) is it possible to arrive at two different answers according to whether one uses the annual or lump-sum method of analyzing costs and benefits? Obviously the uncertainty is due to the rate of interest adopted. The 5% used in this example, for combined interest and amortization, corresponds reasonably to the rate of interest the federal government pays for borrowed money and an assumed 50-yr life of the project. Manifestly, some other rate of interest must be used if the annual and lump-sum methods of analysis are to be brought into agreement. Again, what rate?

Perhaps there is a fallacy involved in crediting the entire value of flood damages to benefits. By so doing the engineer assumes in effect that, had the floods been prevented, the farmer would have gained his customary return, whereas in fact he might have lost the crop anyway by reason of other causes such as drought, insects, glutted market, etc. It is even conceivable that a flood could benefit him in some years by wiping out early in the year a crop which if saved from floods might later have been destroyed by other causes.

The writer has searched in vain for a formula that will determine whether or not a project is meritorious or in fact complies with the mandate of Congress in the Flood Control Act of 1936 that the "Federal Government should improve or participate in the improvement of navigable waters or their tributaries * * * for flood control purposes if the benefits to whomsoever they may accrue are in excess of the estimated costs * * *." Although the ratio of cost to benefit is ordinarily computed on an annual basis using customary interest rates, it is believed that the final determination cannot depend on this ratio alone. A comparison of the capitalized benefits must also be computed and compared with the total cost of the improvements (including capitalized maintenance and operation) before the engineer can be sure that the investment is a good one.

AMERICAN SOCIETY OF CIVIL ENGINEERS

Founded November 5, 1852

DISCUSSIONS

METHOD OF PREDICTING THE RUNOFF FROM RAINFALL

Discussion

BY FRANKLIN F. SNYDER, JUN. AM. SOC. C. E.

FRANKLIN F. SNYDER,[13] JUN. AM. SOC. C. E.[13a]—A concise description of detailed studies of rainfall and runoff relations is presented in this paper. The methods followed give computed values of runoff that agree quite favorably with the observed values for the storms studied.

The methods of analysis followed and relationships developed are quite similar to those used in the Pennsylvania hydrologic investigations[14] for development of flood forecasting procedures and used by the writer[15] to compute a year's record of runoff from rainfall for a 1,147-sq-mile drainage area with no reference to observed stream flow. The authors have used pan evaporation as an index of soil moisture variation, whereas time and temperature were used for that purpose in the studies just mentioned.

There is one striking difference either in conception or in nomenclature: The writer believes that the author's "surface loss," which consists of by far the greater part of the difference between rainfall and runoff for the storms listed in Table 1, is really accretion to soil moisture and that the loss classified as addition to field moisture is probably an initial or surface loss. The most convincing indication of this is a comparison of pan evaporation and surface loss for periods of time for which complete data are given in Table 1.

For all except four of the storms listed the surface loss (Col. 11) is greater than the values in Col. 6 (0.9 $\times$ accumulated evaporation). For the period including the storms of January 4, 1936, to February 17, 1936, the total surface loss is 2.77 in. and the total evaporation is 0.70 in. $\left(\frac{0.63}{0.9}\right)$. Surface loss is

NOTE.—This paper by Ray K. Linsley, Jr., and William C. Ackermann, Juniors, Am. Soc. C. E., was published in June, 1941, *Proceedings*. Discussion on this paper has appeared in *Proceedings*, as follows: October, 1941, by Bertram S. Barnes, Assoc. M. Am. Soc. C. E.; and November, 1941, by Messrs. Richard van Vliet, and LeRoy K. Sherman.

[13] Associate Hydr. Engr., U. S. Weather Bureau, Washington, D. C.

[13a] Received by the Secretary November 14, 1941.

[14] "Report of Cooperative Hydrologic Investigations," by Pennsylvania Dept. of Forests and Waters, U. S. Geological Survey, and U. S. Weather Bureau, Commonwealth of Pennsylvania, August, 1939.

[15] "A Conception of Runoff Phenomena," by Franklin F. Snyder, *Transactions*, Am. Geophysical Union, 1939, Pt. 4, pp. 725–738.

defined in the paper (see heading "Theory: (1) Surface Loss") as "that part of the rain which is intercepted by the vegetal cover and natural or artificial retention basins from which it eventually evaporates and is thus prevented from entering the stream channels." It is difficult to understand how this surface loss by evaporation can be four times the observed evaporation from a pan or how there can be enough natural or artificial surface retention basins to store 2.07 in. (2.77 − 0.70) for later evaporation. For a longer or a more rainy period, the procedure followed might call for a still larger amount of surface storage. The writer believes that a considerable part of this rainfall became soil moisture.

In 1937 the writer made extensive studies of rainfall and runoff relations for storms occurring in the period October, 1934, to September, 1936, on the Hiwassee River Basin above Reliance, Tenn., which includes the Valley River studied by the authors. Using what seemed to be rational values of evaporation and transpiration, a maximum value of 4.0 in. of accumulated soil moisture above a base of 0.00 was obtained in March for the winter of 1934–1935 and 7.5 in. in February for the winter of 1935–1936. At the time it was concluded that the water went to soil moisture rather than deep seepage since the method of computing evaporation and transpiration reduced the accumulated values of soil moisture to the assumed base of 0.00 in. in what appeared to be a reasonable manner during the succeeding summers. These values of variation in soil moisture are much greater than the maximum possible value of 0.80 in. field-moisture deficiency assumed by the authors. This value of 0.80 in. would appear to be a more reasonable maximum value of surface loss.

There is also some question as to the general applicability of the empirical value of 0.05 in. per hr maximum possible absorption. This criterion controls the values of estimated absorption for seven of the first eight storms listed in Table 1, all in February or March of 1935, and does not control in any of the other storms. Better results would have been obtained if the assumed maximum rate of absorption had been larger for the first three storms concerned (the storm of February 14, 1935, is not involved) and smaller for the last four storms. This indicates that the rate decreased with a continuation of frequent rains.

The question raised as to nomenclature and definitions does not invalidate the very good results obtained by the authors, but it would be of considerable importance to someone trying to apply the procedures to other areas.

AMERICAN SOCIETY OF CIVIL ENGINEERS

Founded November 5, 1852

DISCUSSIONS

PIPE-LINE FLOW OF SOLIDS IN SUSPENSION
A SYMPOSIUM

Discussion

By H. A. Einstein, Assoc. M. Am. Soc. C. E.

H. A. Einstein,[9] Assoc. M. Am. Soc. C. E.[9a]—The problem of determining the pump and flow characteristics of a mixture of water and granular solids is of great practical importance. Only the abundance of difficulties involved may explain why so little research has been done and so few papers published in this special field. The numerous complications may also be the reason why the reports of earlier investigators failed to give full descriptions of all technique and apparatus. Both authors stress this fact but omit some information from their own reports. The writer misses especially a short description of the methods of making measurements.

In the pump test, it would be interesting and important to know if volumetric or venturi measurements were used for the discharge and how the two methods compare. For sufficiently fine particles, the venturi meter should give the correct discharge, if the pressure difference is measured by the height of a column of the mixture. For coarser particles, a relative movement of the particles against the water is to be expected in the venturi meter as well as in the pump. The solid particles will probably lag when the acceleration in the venturi meter occurs. Therefore, the water will be accelerated more than the total mixture as an average, and a slightly high manometer reading may be expected. Mr. Fairbank may be able to give some information about this point in his final discussion.

Another important measuring detail pertains to the lines between the taps in venturi, pipe, or pump, and the manometers or piezometer tubes. Part of these lines will be filled with a water-sand mixture, part with clear water. Are the points known at which this change occurs in each line? What correction for any differences in their level has been made? These questions are especially important for the measurement of the "hydraulic gradient S" in Table 3 of Professor Wilson's paper.

Note.—This Symposium was published in October, 1941, *Proceedings*. Discussion on this Symposium has appeared in *Proceedings*, as follows: October, 1941, by Arthur L. Collins, Assoc. M. Am. Soc. C. E.

[9] Hydr. Engr., SCS, U. S. Dept. of Agriculture, Greenville, S. C.

[9a] Received by the Secretary October 29, 1941.

There is a simple device that can be used for measuring discharge for almost all flows through pipes. This device is the measuring elbow. Any elbow following a sufficient length of straight pipe can be used. The dynamic pressure in certain parts of the elbow seems to be extremely stable and therefore can be used for the measurement. The writer found that the point b (see Fig. 14), where the axis of the preceding straight pipe intersects the wall of the elbow, is very reliable. The difference in pressure between point b and a reference point a in front of the elbow is proportional to the velocity head $\frac{v^2}{2g}$ for wide ranges of velocities. For most normal elbows this coefficient seems to be close to unity. This principle of measuring can be used in almost every line of any size without additional energy loss, provided there is an elbow in the line. Here, too, a correction may be necessary for coarse sand in suspension.

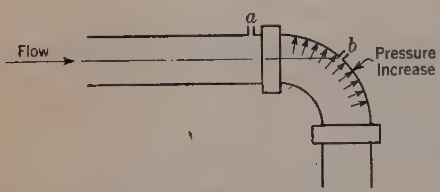


FIG. 14.—MEASURING ELBOW

Eq. 19 is the backbone of Professor Wilson's paper and, as such, perhaps is not sufficiently explained. It seems to be one of those hydraulic rules that simply add energy losses without regard to their interdependence. Its first term represents the amount of mechanical energy dissipated in a flow without the solids. The second term is the amount of turbulence energy dissipated by lifting the solid particles. The two terms, therefore, originate in two altogether different phases of the problem. The same energy may appear in the first term as mechanical energy being transformed into turbulence, and again in the second term, when this same turbulence is dissipated in lifting particles. The measurements themselves seem to indicate this tendency. The scatter of the points of Fig. 11 is considerable, but, still, some groups of points (series 1, runs 1–6; series 1, runs 8–13; and series 2, runs 1–7) unmistakably follow a slope of less than 45°. According to Eq. 19, this can be explained only as an efficiency above 100%, which is impossible. The writer's own experiments, therefore, seem to indicate that Eq. 19 does not explain the problem, even if it does seem to describe it satisfactorily.

Professor Wilson is definitely correct when he calls the criterion for settling of the sediment on the bottom of the pipe "a bed-load problem." It is definitely to be solved as a problem of the stability of this deposit. Unfortunately, present knowledge about bed load does not include any flows that could be compared with the flow in one of these pipes. Therefore, caution must be used when data from rivers are applied to pipes. However, if all the necessary information were available, these and similar pipe measurements could very well give exceedingly important information pertinent to certain bed-load problems. However, at least one series of measurements with 100% open cross section should be included, and, as stated previously, the measurements themselves should be described in more detail.

Both papers of this Symposium will prove to be very helpful to the engineer who must design apparatus for pumping solids in suspension. They provide him with some necessary information in a convenient and sufficiently accurate form.

AMERICAN SOCIETY OF CIVIL ENGINEERS

Founded November 5, 1852

DISCUSSIONS

PILE-DRIVING FORMULAS

PROGRESS REPORT OF THE COMMITTEE ON THE BEARING VALUE OF PILE FOUNDATIONS

Discussion

BY MESSRS. TRENT R. DAMES AND WILLIAM W. MOORE, MAXWELL M. UPSON, GREGORY P. TSCHEBOTARIOFF, ROBERT F. LEGGET, AND JACOB FELD

TRENT R. DAMES,[41] ASSOC. M. AM. SOC. C. E., AND WILLIAM W. MOORE,[42] JUN. AM. SOC. C. E.[42a]—This Report is noteworthy because of its exceptionally concise and fair-minded presentation of a broad and highly controversial subject. The writers believe the viewpoint and emphasis of Report *B* represent more nearly a proper evaluation of the relative importance of dynamic pile formulas, static pile formulas, and pile loading tests than those presented in Report *A*.

Dynamic Formulas.—In recent years the use of double-acting steam hammers has become very common and drop hammers are only rarely used. As this trend must be expected to continue, discussions of dynamic formulas that are restricted to drop hammers and single-acting steam hammers will be of limited value at best. Also, it is not clear how h_0 would be determined for single-acting steam hammers for use in Eq. 17.

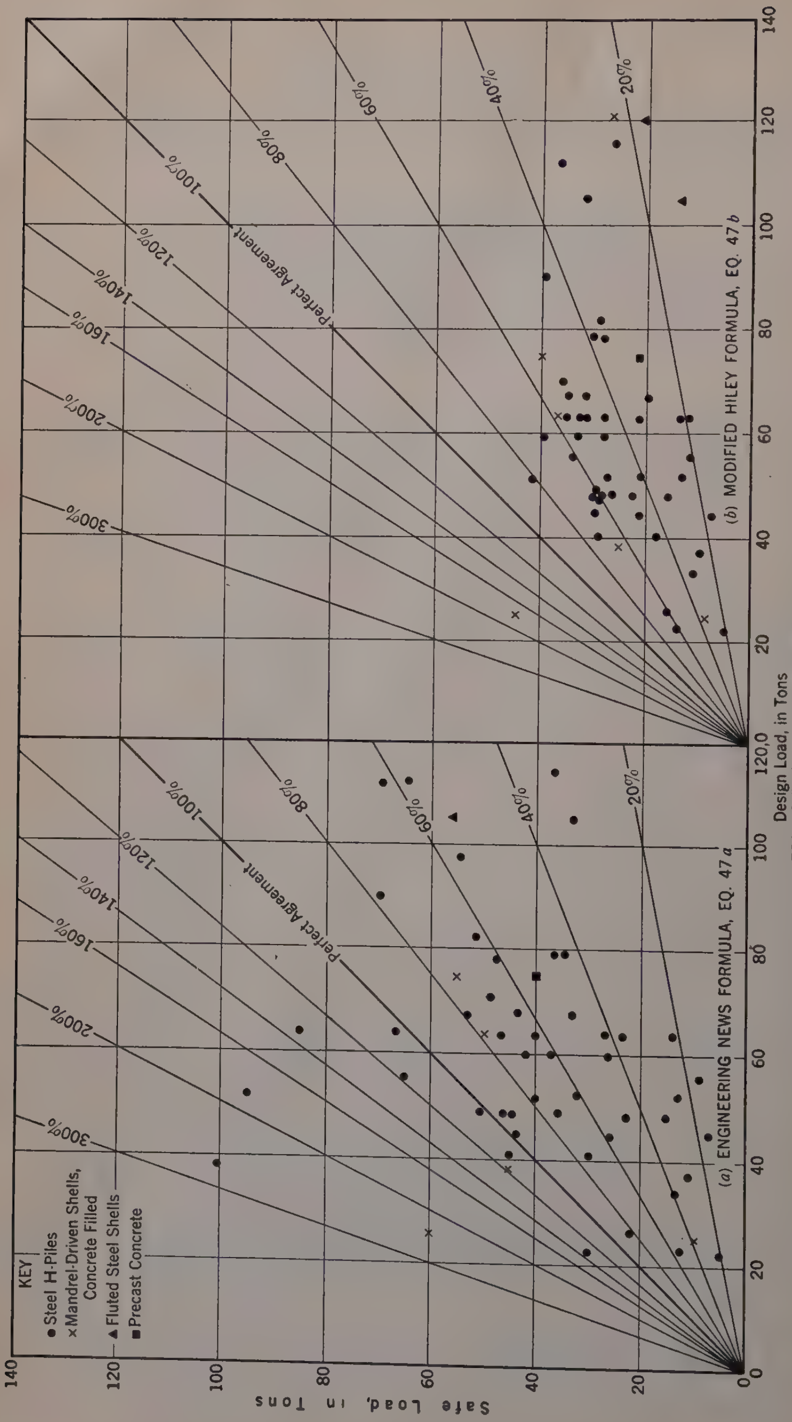
As a test of the validity of driving formulas, the results of certain loading tests made under the supervision of the writers (including some made by the U. S. Engineer Department in the Los Angeles River Channel and at Sepulveda Dam under the direct supervision of the junior writer while employed by the government) have been compared with two dynamic formulas. The charts of Fig. 5 show a comparison between a "design load," taken as 75% of the yield-point capacity determined by load tests, and "safe loads," given by both the

NOTE.—This Report was published in May, 1941, *Proceedings*. Discussion on this paper has appeared in *Proceedings*, as follows: September, 1941, by Messrs. G. G. Greulich, C. O. Emerson and D. O. Northrup, Harry J. Engel, and John D. Watson; October, 1941, by Messrs. Robert D. Chellis, Lazarus White, John G. Mason, Carlton S. Proctor, George Paaswell, and Abraham Woolf; and November, 1941, by Messrs. Howard T. Evans, William G. Atwood, Donald M. Burmister, Wallace E. Belcher, Clement C. Williams, and D. P. Krynine.

[41] (Dames & Moore), Los Angeles, Calif.

[42] (Dames & Moore), Los Angeles, Calif.

[42a] Received by the Secretary October 29, 1941.



Radial Lines Are the Percentages of Actual Safe Design Load Allowed by Dynamic Formula

FIG. 5.—COMPARISON OF DRIVING FORMULAS WITH LOAD TESTS

"Engineering News" formula—

$$R = \frac{2\,W\,h}{s + 0.1} \quad \text{.......................(47a)}$$

and a modified form of the Hiley formula—

$$R = \frac{A\,E}{8\,L}\left[-s \pm \sqrt{s^2 + 4\,W\,h\,\frac{W + n^2\,P}{W + P} \times \frac{L}{A\,E}}\,\right] \quad \text{......(47b)}$$

which is used in the Uniform Code of the Pacific Coast Building Officials Conference. In Eq. 47*b* all loads are in pounds and lengths in inches. Piles driven with both single-acting and double-acting steam hammers are represented. In general, the dynamic formulas show quite low capacities, which means that excessive quantities of unnecessary piling are used in cases where these formulas are relied upon. However, the "Engineering News" formula gave unsafe loads in 16% of the cases investigated; the modified form of the Hiley formula gave one unsafe value. The scatter of data is so wide that the only conclusion possible is that the dynamic formulas are unreliable and, in most cases, are likely to lead to unnecessarily expensive construction costs.

Static Formulas.—It was stated in Paragraph *A*-13 that actual tests show that the pressure on the side of a displacement pile does not increase directly with depth. The writers would be extremely interested in knowing more about the details of these tests. Do they include pressure cell measurements? Although the soil pressure on the side of a pile may not increase in direct proportion to the depth, it would be reasonable to expect the pressure to increase in proportion to the shearing strength of the soils, thus enabling these pressures to be at least approximately evaluated.

In this connection, the writers have developed a procedure which has been used to calculate the bearing capacities of about $2,000,000 worth of different types and lengths of piling since 1939. The calculations are based on the results of tests on core samples, which tests determine the shearing strength and the friction of the penetrated soils on the particular pile materials proposed. It is assumed that the failure of a pile may occur as the result of either a shearing failure in the surrounding soil, a frictional failure on the surface of the pile, or a combination of both, whichever is least.

For piles of small displacements, such as steel H-piles, it is assumed that the lateral pressure on the surface of the pile is equivalent to the natural overburden of the soil mass. In the case of displacement piles, during the driving of which the soils are pushed aside and remolded or compacted, it is assumed that the lateral pressure on the surface of the piles is equal to the ultimate permanent passive pressure of the soil as limited by its shearing strength. This passive pressure is taken as equal to a quantity between π and $\pi + 2$ times the shearing strength, depending upon the character of the soil.

The results of the calculations based on this method of analysis are usually presented in the form of curves showing the lengths of various types of piles required to carry a range of loads; Fig. 6, prepared for a site in Pittsburg, Calif.,

illustrates such a set of curves. The factor of safety to be selected for determining the design loads from these curves depends upon the type, flexibility, and importance of the structure, as well as upon the extent and completeness of the investigations and the uniformity of the soil conditions.

The accuracy of the capacities calculated by this method has repeatedly been verified by loading tests to failure. Figs. 7, 8, and 9 illustrate three such loading tests performed on a site in San Diego, Calif. Shown are the test setup and the soil conditions (Fig. 7), the driving resistance of the test piles (Fig. 8), and the load-settlement curves obtained (Fig. 9). The test loads were applied by mechanical jacks and measured with pressure cells; and the settlements were measured with micrometer dial gages. Single-acting steam hammers were used in each case (weight of ram, 5,000 lb, and stroke, 36 in.), the energy per blow being 15,000 ft-lb. The capacities (computed some time before the per-

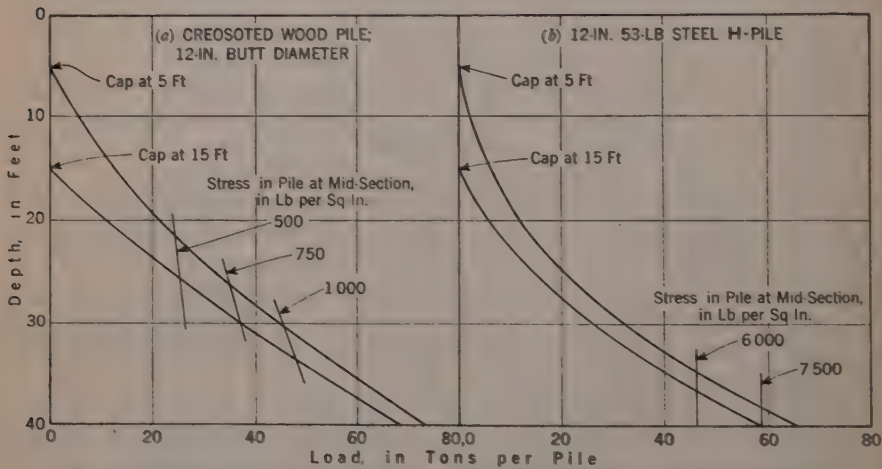


FIG. 6.—BEARING CAPACITY OF DRIVEN PILES (CURVES REPRESENT YIELD-POINT VALUES)

formance of the loading tests from analyses of the shear and friction tests) are indicated on Fig. 9 so that the yield-point bearing capacities determined by the field-loading tests may be compared with the calculated capacities. In computing the design capacities the temporary support of soft soils was taken into account, and the curves of safe design capacity made allowance for the consolidation of soft soils.

It is mentioned in Paragraph A-14 that methods of obtaining friction values "have already been described." This description cannot be found by the writers, in the Committee's Report, but it is assumed that the methods probably involve field-loading tests or "pull-out" tests. In Table 1, the material and shape of the pile and the intervals between driving and testing should be given. The reported values of "*f*" give average shear or friction values (or both) for the particular type and length of pile tested under the particular local field conditions. These values are not directly comparable to friction tests on core samples.

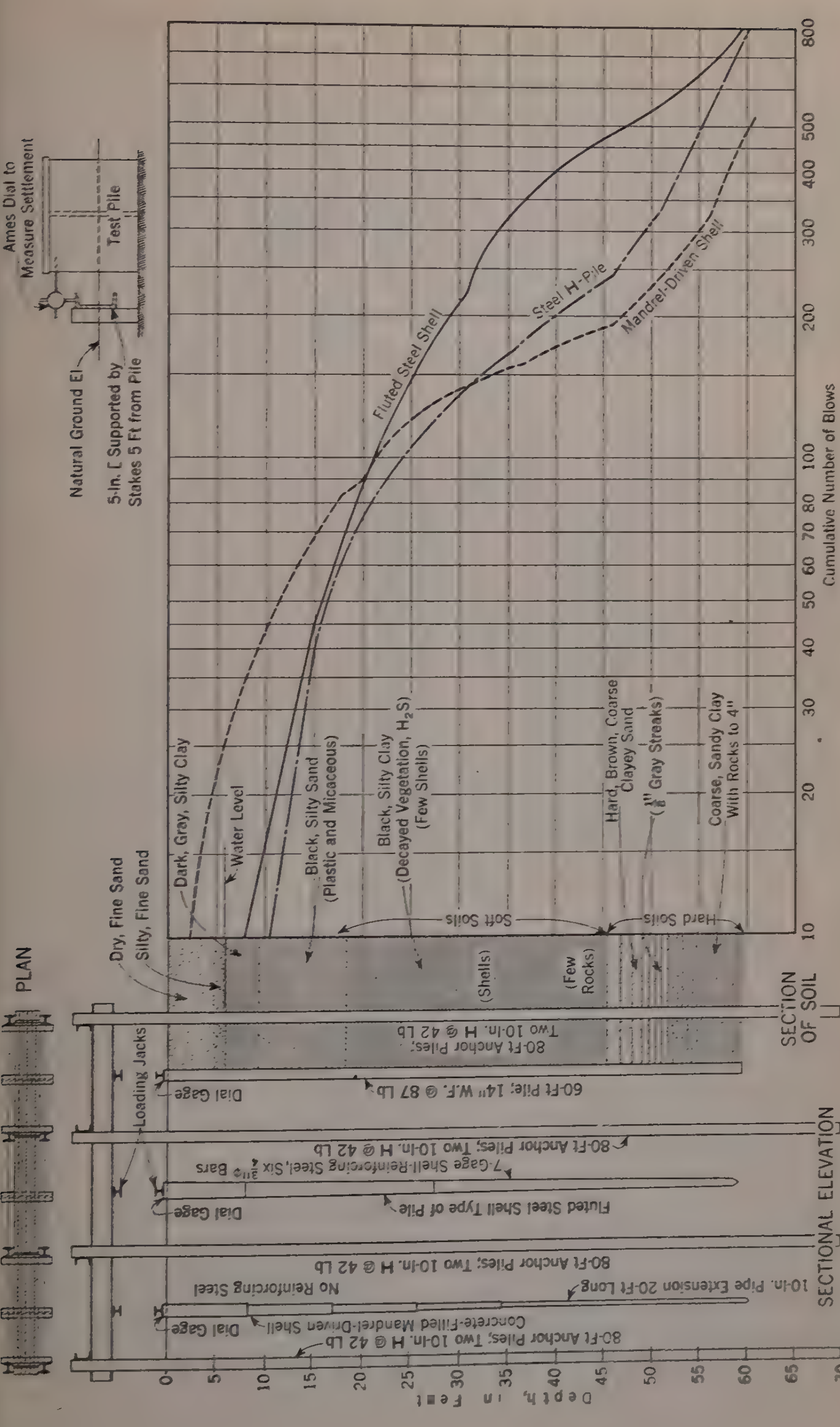


FIG. 7.—TEST SETUP AND SOIL CONDITIONS

Test Piles, Tests, and Service Records.—As a loading test on a pile can measure only the capacity of the pile to carry load before slipping relative to the supporting soil, the specific magnitude of the settlement measured in a load test is of little or no direct value. It is essential that a complete load-versus-settlement record be kept using small increments of load from zero to failure and that the time-rate of settlement of the pile under each increment of load, and also the additional increments of settlement with repeated rebounds (that is, the removal and reapplication of a given test loading), be known. In this way only is it possible to ascertain the load that first causes progressive settlement. Accurate measurements of both the rate of deflection and the loads enable the completion of a test within a reasonable length of time and permit the plotting of smooth test curves that can be interpreted reliably.

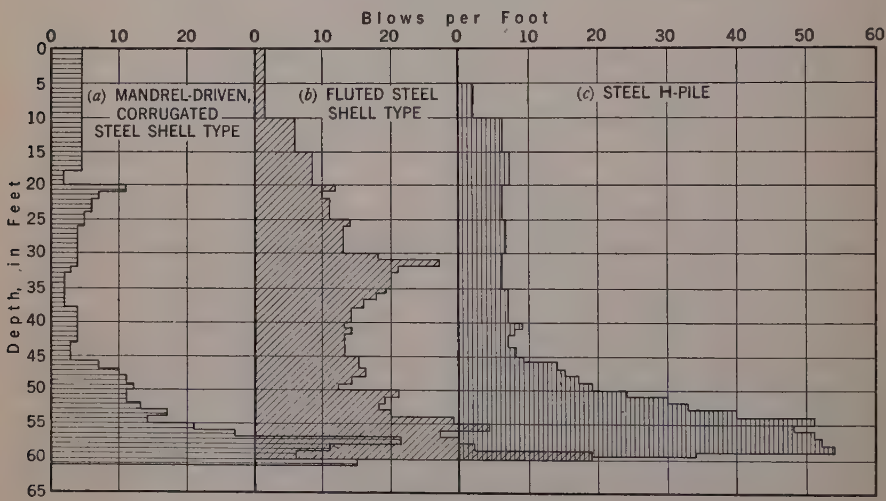


FIG. 8.—DRIVING RESISTANCE

The results of dead-weight tests are often difficult to interpret because: (1) The vibration and impact during the application of the loads usually result in a more or less irregular and ragged test curve; (2) the tendency to apply the loads in large increments results in only a few test points being available for plotting a complete curve; and (3) the weight of the material that must be moved is so great that it is very seldom economically possible to obtain a number of rebound tests sufficient to ascertain the behavior of the pile under repeated loading.

Hydraulic jacks operating against an adequate reaction may prove satisfactory, but because of the frictional drag between the jack cylinder and the piston and cup washer, the calibration is seldom reliable for movements requiring both extension and retraction of the jack. The use of mechanical jacks, combined with accurately calibrated load-measuring gages and using either dead loads or anchor piles for the jack to react against, has been found by the writers to be much more satisfactory than the use of either dead weights or

hydraulic jacks. Anchor piles have been spaced at distances varying from 3 ft to 12 ft from the test pile with no perceptible differences in test results.

The use of micrometer dial gages on reference bars is believed to offer the best means of obtaining accurate settlement measurements. Although the use

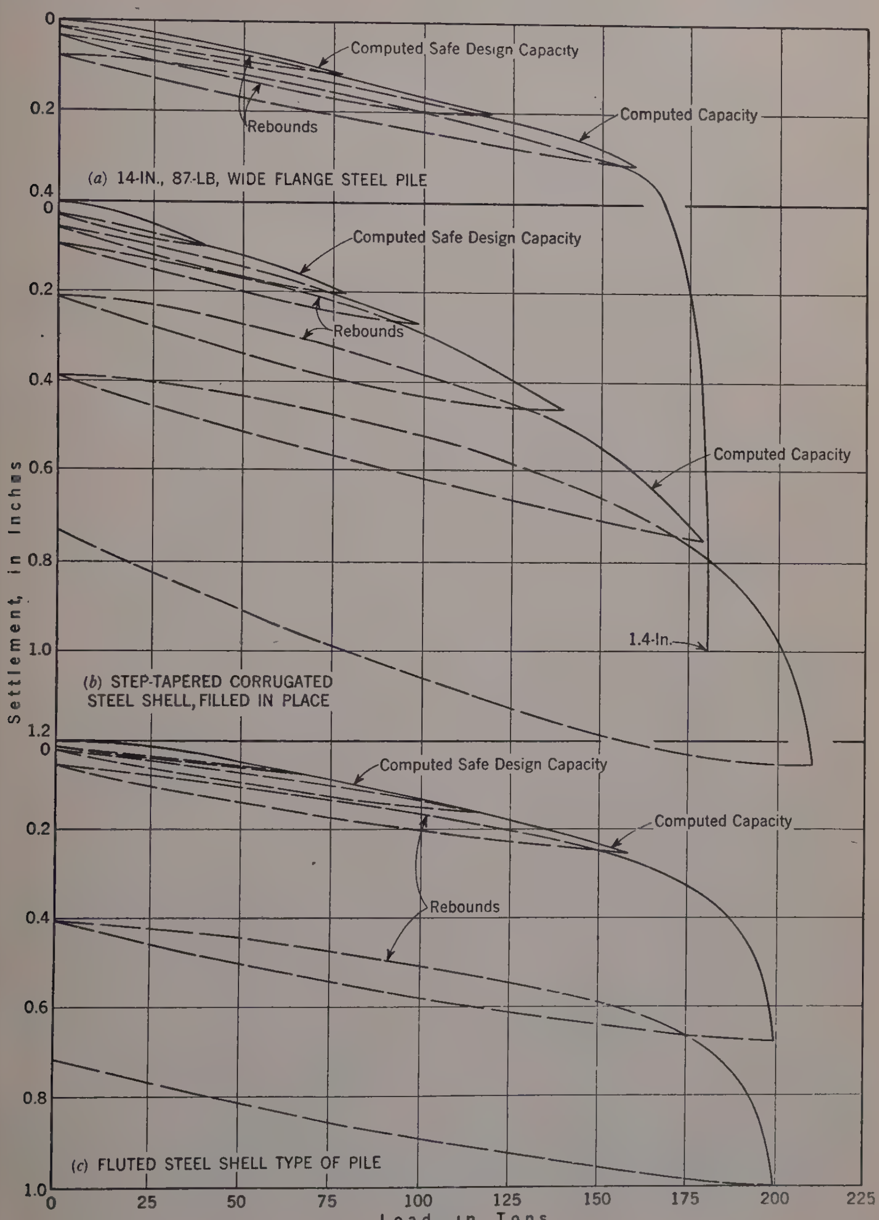


FIG. 9.—LOAD-SETTLEMENT CURVES (LENGTH OF PILES, 60 FT)

of an engineer's level as a check on any disturbance of the reference bar is desirable, no appreciable disturbances have been observed by the writers except on very soft soils.

It is believed best to maintain each load increment until the settlement stops, or until a definite progressive settlement indicating the failure of the pile is observed, rather than to increase the load at arbitrary time intervals. When a test must be completed in such a short time that arbitrary time intervals are necessary, it is best to maintain uniform or geometrically increasing time intervals (for uniform load increments) during the entire duration of the test in order to obtain a smooth and continuous test curve from which a yield point may be selected. If the time intervals are varied irregularly for different loads, the resulting test curve is apt to be so irregular that no definite interpretation can be made.

Fig. 9 demonstrates that the load which caused a permanent settlement of 0.01 in. per ton (the criterion mentioned in the Report for determining the allowable load from a load test), is far in excess of the safe bearing capacity or yield-point strength of any of these piles. The application of this rule of thumb would permit the use of from 180 tons to 220 tons per pile, whereas the piles are only safe for loads of from 50 to 60 tons each.

An example in which 0.01-in. settlement per ton of load determines an unsafe bearing capacity has been offered by G. P. Tschebotarioff,[43] M. Am. Soc. C. E. The load-versus-settlement curve, for the pile test shown, signifies that a yield point in the test curve probably occurred at a load of not more than 40 tons per pile, even though the total settlement was less than 0.01 in. per ton at the design load of 60 tons. The building constructed on these piles was observed to have settled more than 10 in.

On the other hand, in the case of a field loading test performed on a driven cast-in-place concrete pile, as part of a foundation investigation on a site in Compton, Calif., this criterion would have restricted the design load to practically zero. The calculated yield point load was 30 tons, the yield point of the load test was approximately 33 tons, a design load of 20 tons was used, and the foundation is performing satisfactorily.

Although the value of 0.01 in. per ton might be satisfactory as an approximate indication of the capacities of certain piles in certain soil conditions, it has been found unusable by the writers who are completely convinced that a load test, to be interpretable, must be performed in a manner sufficiently accurate to permit the determining of a yield point from the shape of the curve, as is done in tests of structural materials.

This discussion has been limited to the subject matter of the Committee's Progress Report which is, in effect, the determining of the bearing capacity of a single, isolated pile. It is hoped that the Committee will eventually accomplish this first step and then proceed to the solution of the main problem—the supporting capacity of pile foundations composed of groups of piles.

[43] "Settlement Studies of Structures in Egypt," by Gregory P. Tschebotarioff, *Transactions*, Am. Soc. C. E., Vol. 105 (1940), Fig. 14, p. 937.

MAXWELL M. UPSON,[44] M. AM. SOC. C. E.[44a]—It is the writer's understanding that the object of this Committee is to provide a Manual of Engineering Practice for engineers engaged in pile driving. It would seem that, in order to attain this end effectively, the simplest possible formulas and information should be advocated.

The authors of Report *A* seem to have deviated widely from this principle. It is interesting, of course, to those who desire to go into involved mathematics to develop formulas based on assumptions; but in the writer's opinion the place for this material is in a textbook rather than in a Manual of Engineering Practice. If this principle is accepted, most of the equations in Report *A* should be eliminated, since they may serve to confuse those who are not skilled in pile-driving engineering. If the Committee wishes to recommend a particular pile-driving formula, or possibly several formulas, it should publish them in a form in which experience has demonstrated they could normally be used in connection with the pile-driving operation. Each formula should be accompanied by a clear statement of its usefulness and limitations. In order to present the material in the Report in a dependable manner, it would seem that there are several statements in Report *A* that should have further consideration by the Committee.

Report *A* indicates to the average reader that there is a desire to "sell" Eq. 9 to the engineering profession, whereas it is well known that the "Engineering News" formula (Eq. 6) is the most widely used and generally accepted. There is the implication in Paragraph *A*-6 that a perfectly elastic impact is "an unwarranted assumption." In Paragraph *A*-8, the assumption of a perfectly inelastic impact is made in the derivation of Eq. 9, and this is subsequently referred to in Paragraph *A*-12 as a "reasonable assumption." The fact is that the impact in pile driving is neither perfectly elastic nor perfectly inelastic, but is somewhere between the two extremes. If it is "reasonable" to assume one extreme, it is equally reasonable to assume the other. In either case, the principal purpose of the assumption is to simplify the final formula.

This same type of inconsistency is evidenced in the last sentence of Paragraph *A*-6, which implies that "substituting constants for widely varying factors" in Eq. 5 is wrong. In Eq. 9 the values of k are determined by Eq. 11. In this there are three factors and these refer to the cushion block, the pile, and the soil. It is well known that all three of these factors that occur in pile-driving operations vary over a wide range. In consequence, the constant used in Eq. 9 may prove most misleading, since it is labeled a "reasonable assumption" in Paragraph *A*-12. Against this recommendation a similar procedure in Paragraph *A*-6 is condemned.

It is the writer's apprehension that the presentation of complicated formulas such as Eq. 9, requiring so many assumptions, may well lead the uninitiated engineer astray. It would seem that in Report *A* as written the Committee is trying to prove that Eq. 9 is the only pile-driving formula that is based on reasonable assumptions. Actually there are as many doubtful and unwar-

[44] Pres., Raymond Concrete Pile Co., New York, N. Y.

[44a] Received by the Secretary October 31, 1941.

ranted assumptions in Eq. 9 as there are in any other pile-driving formula in existence. The real danger in the use of pile-driving formulas like Eq. 9 lies in the fact that the formula looks authoritative because it appears to take into account all the variable factors that are involved.

Those experienced in pile driving know that, under certain conditions, Eqs. 8 and 9 may be very misleading. If the weight and rigidity of the driven member are not sufficient to avoid lateral vibration and to carry the impact stresses down to the material to be penetrated, the actual penetration per blow is of little value as an evidence of the carrying power of the pile. This vibration or oscillation is so serious under some ground conditions that light-weight piles show less penetration per blow than much heavier piles of sufficient strength and mass to resist oscillation and to carry the effectiveness of the blow to the resisting underground material. These facts completely contradict the "$W + P$" found in Eqs. 8 and 9.

It is the experience of the company with which the writer is associated that when extremely hard lenses of material are encountered that must be penetrated, it is necessary to use a core or driving medium of considerably heavier weight than the moving part of the hammer. In carefully made comparative tests—driving pipes 100 ft long—it was found that the slight difference between a pipe 0.307 in. thick as against 0.259 in. resulted in the penetration of the 0.307-in. pipe into the hardpan about 3 ft farther than the 0.259 in. This resulted in increasing the allowable load on the thicker pipe some 20%.

Expressing the principle in another way, in some cases the resistance of the pile under the hammer is less for the heavy pile than for the light pile which (as indicated previously) is directly contrary to the principle set forth in Eq. 9. This principle may well be understood when applied to the driving of a very light-weight shell on which the blow is transmitted to the thin shell itself. An examination of motion pictures shows clearly the oscillation of the lighter shell under the impact of the hammer absorbing a part of the blow.

This may be a new concept to some engineers, but it can be understood readily by recalling the vibration of a wood pile when it is being driven through a hard stratum near the surface, or a thin steel shell when unsupported or meagerly supported for a large part of its length. It is obvious that the vertical blows are dissipated in lateral vibration and bending, so that comparatively little force is conveyed to the material that is to be penetrated. This principle undoubtedly has been the cause of some of the settlements of light steel shells that have been driven to a final resistance normally expected to have assured satisfactory carrying capacity.

Almost two-score years of experience, with difficult pile-driving operations all over the word, have convinced the writer that pile driving is by no means an exact science. This experience has developed almost no instances of failure of piles driven to the requirements of the "Engineering News" formula, in which good common-sense methods have been used. The few settlements that have occurred were due to other factors that had not been taken into consideration—namely, piles of inadequate structural rigidity or strength; lack of borings which would have disclosed underlying soft strata; excessive

loading of the topsoil surrounding the piles; the draining away or the loosening of the penetrated material by adjacent operations; and the complete ignorance or omission of soil analysis.

The art of pile driving cannot be reduced to a problem of mathematical equations. It is the development of practical experience and sound judgment. For that reason, it is hoped that the Committee will recognize this fact, so that the proposed Manual will present the problem of pile-driving dynamics in a manner that will be simple and practicable to the practicing engineer.

Gregory P. Tschebotarioff,[45] M. Am. Soc. C. E.[45a]—The Report reflects the present diversity of views on the subject of pile-driving formulas. It provides a good basis for a discussion of this controversial field and should stimulate a critical attitude in a domain where so much has been blindly accepted in the past.

In its present form the Report is essentially a compromise, although two separate reports have been submitted. Each of them (especially Report *A*) appears to have been influenced to some extent by the point of view expressed by the other. As a result, non-specialists seeking to find guidance in this Report may receive a rather confusing general impression. Therefore, it is to be hoped that a non-ambiguous set of guiding rules may result from a merging of the numerous useful points contained both in Report *A* and in Report *B*. With this end in mind, the writer offers the following critical suggestions concerning such possible merging:

(1) In the writer's opinion, a proper evaluation of the function and purpose of a pile-driving formula is given in Paragraph *B*–6:

> "* * * any dynamic pile-driving formula is nothing more than a yardstick to help the engineer secure reasonably safe and uniform results over the entire job. The use of a complicated formula is not recommended since such formulas have no greater claim to accuracy than the more simple ones."

Paragraphs *B*–1 to *B*–5 are sufficient to clarify this statement and should all be retained in the final report with perhaps only a few additions of minor importance—for instance, some items from Paragraph *A*–3.

Report *A*, on the other hand, has no clear-cut general approach to the foregoing point, which is one of primary importance. Thus, Paragraph *A*–2 recognizes that all pile formulas are "approximations" and subject to "certain limits and conditions." It further states: "Unless these limits and conditions are thoroughly understood * * * the use of a pile formula may be dangerous." Yet it does not specify in detail what the limits and conditions are, the knowledge of which may help engineers to avoid danger. Paragraph *A*–3 briefly mentions a few, but does not nearly exhaust all of the possibilities. This paragraph may even be misleading in its implication that "perfect" dynamic formulas may exist. Furthermore, the undue emphasis placed on various individual dynamic formulas in paragraphs *A*–4 to *A*–12 tends to strengthen this erroneous impression. Therefore, in the writer's opinion, paragraphs *A*–4

[45] Soils Consultant—Moran, Proctor, Freeman & Mueser, Cons. Engrs., New York, N. Y.; Asst. Prof Civ. Eng., School of Eng., Princeton Univ., Princeton, N. J.

[45a] Received by the Secretary November 5, 1941.

to A–12 could well be discarded in favor of Paragraph B–10, which recommends field measurements of elastic and other coefficients for use in any formula in preference to office calculations of these quantities. Paragraph B–10, however, might well be expanded to cover the methods of such field measurements.

(2) The title of Report A is "Pile Formulas," and that of Report B is "Pile Formulas and Pile Tests"; yet paragraphs B–13 to B–15 provide much less data on the subject of actual pile testing than do paragraphs A–16 to A–23. Therefore, this latter part of Report A is to be recommended for inclusion in the final draft with some additions and changes.

The importance of precise instructions concerning details of pile-testing procedures cannot be overemphasized. For instance, the writer knows of a case in Egypt in which a contractor placed 200 tons on a 27-ft long single cast-in-place pile that was supported by a deep bed of stiff plastic clay. Under that load, zero settlement was then claimed, measured to 0.01 in. It was discovered, however, that a lever indicator, which was supposed to magnify the actual settlement, had its fixed support within a few inches of the skin of the pile–obviously settling together with the pile. This does not appear to be an isolated case. Thus, contrary to the very proper instructions of the present Paragraph A–22 concerning this point, an important and reputable museum in a large city of the Middle-Atlantic seaboard in the United States displayed a "model" test pile in 1938, the settlements of which were supposed to be measured by an Ames dial placed on the ground surface next to the skin of the pile.

(3) Paragraph A–14 gives data for the evaluation of the frictional support of piles. The danger of relying on friction where more than one pile or a small group of piles is concerned should be emphasized at this point. Most present static formulas arbitrarily deal with friction, cohesion, and point-bearing resistance, which are assumed to have fixed values for a definite type of soil. These formulas do not attempt to relate these values to the complicated shearing and compressive deformations in the soil around and beneath the pile.

The writer observed the following extreme case that will illustrate this point: A 45-ft long, cast-in-place concrete bore pile was sunk through 10 ft of fill and 30 ft of stiff clay into a fine silty sand layer. Comparative compressive strength tests on undisturbed samples of this clay showed that it was not unfavorably affected by remolding. The sand layer was fairly loose originally, and the sand below the base of the pile may have been further loosened during the sinking of the bore pile. During a test the load-settlement curve at first had normal characteristics, showing a gradual progressive increase of the settlements (which reached 0.25 in. under a load of 42 tons), when a sudden drop of 1.5 in. was observed. A further increase of loading was accompanied by much smaller settlements than prior to the sudden drop.

What had happened was obviously this: At first the pile was supported merely by the shearing resistance of the stiff clay, which interlocked with the rugged skin of the pile. The settlements were caused by the shearing deformations in the clay, the loose sand at the base offering very little resistance. At a total load of 42 tons the shearing stress along the skin of the pile reached an average value of 350 lb per sq ft, and the pile sheared through the clay, thereby compacting the sand beneath its base and increasing its resistance until

the sand was able to support most of the load. The further smaller settlements with continued loading were then mainly due to compressive deformations in the sand beneath the pile base.

The foregoing extreme case serves to illustrate the following facts:

(*a*) The total load carried by a pile will represent the sum of the supports provided by the soil around the pile through friction or shearing resistance and by the soil beneath the base through resistance to compression.

(*b*) The values of both types of support are directly related to the deformations in the soil and to the settlement of the pile as induced by these deformations.

(*c*) The nature of the relationship between the support provided by a soil and the extent of its deformation will be strongly affected by both the nature and the state of a soil. Specifically, a loose sand beneath a pile base may undergo considerable compression, gradually increasing its resistance. A compact sand will behave differently. It may undergo very little deformation before its resistance reaches a maximum, after which its resistance to compression will decrease due to shear failure under the pile base. A soft plastic clay, especially if further weakened by remolding, may give very little frictional support to a pile, but this support will not vary much with the pile settlement and with the resulting shearing deformation in the clay after a maximum resistance has been gradually reached. On the contrary, a stiff clay, especially if it is of a type not unfavorably affected by remolding, will quickly reach a maximum of its shearing resistance, after which this resistance will decrease rapidly with further deformation. Therefore, the settlement at which the resistance of the base reaches its maximum may be different from that at which the resistance of the soil to shear around the skin of the pile is at its maximum.

(*d*) In most practical cases piles are not embedded in deep homogeneous deposits, and the rigidity of the different soils supporting a pile is not uniform or constant. It follows that a static formula cannot ignore the possible variations in the stress-strain relationship of soils around a pile as this has been done by static formulas so far.

(*e*) The resulting present uncertainties appear to be better expressed by Paragraph *A*–13 than by the corresponding paragraphs *B*–8 and *B*–9.

(4) The statements of both Reports concerning the danger of estimating the bearing capacities of pile clusters from any pile formula or from the results of load tests on single piles should be retained and further emphasized.

The foregoing critical comments are offered as a preliminary attempt to find a common basis for a necessary merging of the two Reports. This merging appears possible since their difference seems to be more in manner of expression than in basically different points of view.

ROBERT F. LEGGET,[46] ASSOC. M. AM. SOC. C. E.[46a]—The publication of this interesting Report is presumably a preliminary step toward the preparation of the Manual of Engineering Practice, referred to by the Committee in the

[46] Asst. Prof., Dept. of Civ. Eng., The Univ. of Toronto, Toronto, Ont., Canada.

[46a] Received by the Secretary November 17, 1941.

"Foreword." In view of the nature of the Committee, this Manual will naturally be restricted to pile foundations. Since pile foundations are only one of many types of foundation structures, it is strange to find no attempt in the Report to describe the relation of pile foundations to foundations in general.

A manual is described by the Technical Procedure Committee of the Society as "an orderly presentation of facts on a particular subject, supplemented by an analysis of the limitations and applications of these facts. It contains information useful to the average engineer in his every-day work * * *." A Manual on Pile Foundations will be very widely welcomed, in view of the importance of foundation work. Surely, therefore, any such Manual must place pile foundations in proper perspective, in relation to foundations as a whole, and not present them as indispensable to foundation practice.

Two factors lend added urgency to this necessary feature of the Manual. Many engineers will know of one or two examples from practice in which the use of bearing piles as foundation units did more harm than good; those who have made a study of foundation practice will know that this misuse of piles is not the isolated occurrence generally supposed; yet the average engineer in general practice does not know this, and therefore should be so advised in any Manual prepared for his guidance. Not only does the average engineer not know this, but he is subjected constantly to the influence of the attractive sales promotion activities of commercial enterprises specializing in some branch of piling work. No exception can be taken to these activities in general; they are mentioned by way of contrast to the fact that the engineer in general practice is very rarely presented with the necessary complementary viewpoint—namely, that there are many foundation problems in the solution of which bearing piles should not be used. If the prospective Manual is to serve its purpose effectively and present a truly objective survey of pile foundations, then this limitation of piles as foundation units would seem to be an essential feature of its contents.

In order to determine whether bearing piles should or should not be used as foundation elements in any design, it is imperative that the nature of the soil at the prospective site be known to a depth of at least twice the width of the structure proposed. Therefore, an adequate program of test borings is an essential preliminary to the decision to use bearing piles in all general foundation problems. This matter is dealt with, very briefly, in the "Foreword" to the Report; it is naturally not mentioned in Report *A* on "Pile Formulas"; somewhat surprisingly it is not mentioned in Report *B* on "Pile Formulas and Pile Tests," even in connection with the useful description therein given of the use of test piles. When it is recalled that the results of a test load on one pile may be invalidated when applied to an assemblage of piles beneath a large structure, because of the presence of a deeply buried weak soil stratum, this neglect of essential subsurface exploration is strange and disquieting.

Proceeding to the next step in logical foundation design, if it is found that bearing piles must be used to support a structure, and that they can be used safely in this manner, then it is necessary to know in design what load can be taken by each pile. The most obvious way of determining this safe load is by

calculation, based on a knowledge of the properties of the soil surrounding the pile and of the manner in which this soil assumes the load. It is not yet possible to make such calculations accurately. An interesting and suggestive approach to this problem in soil mechanics, however, is to be found in a paper published in 1939 by I. F. Morrison of the University of Alberta, in Edmonton, Alberta, Canada.[47] Some work along the same line has been started at the University of Toronto, in Toronto, Ont., Canada, but has inevitably had to be discontinued for the duration of the war.

An alternative approach is to test a full-sized pile, driven into place, with loads appreciably greater than that contemplated for design. This is analogous to the routine testing of all the structural materials that go into superstructures; it is logical; it is economical; it is definite. The latter section of Report *B* is to be welcomed, therefore, as a useful outline of procedure for testing a bearing pile. It is short, and could be expanded profitably before incorporation in any Manual issued by the Society. More attention should be given to the interpretation of the results of a loading test, and much more emphasis placed on the fact that the maximum safe load on one test pile will never be the maximum safe load on single piles driven in clusters in the same soil.

What, then, of pile formulas—for, if the procedure outlined is followed, all necessary data for the design of a pile foundation will have been obtained. There are quite a number of students of foundation engineering who would be content to stop at this point and give no attention to formulas at all. The writer cannot subscribe to this extreme position. (See, for example, Professor Morrison's paper and discussion.[47]) In the first place, it is always desirable, if not essential, to be able to check the penetration of the various piles of any group, especially in relation to the corresponding test pile penetration. Admittedly it is not necessary to have a formula for this purpose if the same driving equipment is used, but a formula is necessary if different pile-drivers are used either together, or for test and service piles respectively. A helpful illustration of this use of pile formulas is on record.[48]

In the second place, in ordinary civil engineering practice, there are a large number of jobs so small in themselves that they do not warrant the expenditure of any money on subsurface exploration provided that local foundation conditions and practices are well known, and known to apply to the jobs in question. On work of this nature, and only on work of this nature, does there appear to be any justification for the use of a pile-driving formula for determining, approximately at best, the safe load that may be put upon a bearing pile. Even this use of pile formulas, if intelligently applied, is so conditioned by necessary precautions and restrictions that it is no "easy way" to safe foundation design. In view of this, attempts to "simplify" the Hiley formula (reflected, to some extent, in Report *A*) appear to be not only unjustified but also unwise.

It will be seen that the writer regards pile formulas as of relatively minor importance in the design of pile foundations. In view of this he purposely refrains from discussing in any detail the recommendations in Report *A*, feeling

[47] "The Fundamentals of Pile Foundations," by I. F. Morrison, *Engineering Journal*, E. I. C., October, 1939, pp. 431–434; discussion, February, 1940, pp. 63–64.

[48] "Steel Sheet Pile Wharf at Rimouski, Quebec," by J. P. Carrière, *Civil Engineering*, December, 1939, pp. 707–710.

strongly that this section of the Report serves merely to bolster up the false importance so generally attached to pile formulas. It was inevitable, perhaps, that two divergent reports should emanate from the Committee. The writer would like to express the hope that, as a result of the public discussion of the dual Report, agreement may be reached in the near future by the relegation of pile-driving formulas to their proper place as useful, but very limited, calculating aids in the construction of some pile foundations, after it has been found that bearing piles can be, and should be, usefully employed as foundation elements.

JACOB FELD,[49] M. AM. SOC. C. E.[49a]—The Committee Report presents two methods for standardizing the design of piles in construction work.

Report *A* recommends the use of a formula and practically orders that static tests of completed piles be required to check the assumed pile values in the design.

Report *B* lists references where the various formulas can be found but makes mandatory static testing of piles as the only determination of their bearing value. Since all formulas are really mathematical expressions of a dynamic test during pile driving, except the formula that is based solely on the theoretical skin friction of a pile, the true difference between the two reports is whether the design of piles shall be based on a dynamic test as checked by the static test, or on the static test alone. Throughout both reports, and especially the "Foreword," there seems to be too much expression of alarm concerning the bearing value of piles. Perhaps this is due to the assumption that piles are foundations. In the "Foreword" there is the expressed statement that the soil is just as much a part of the structural design as is the framework or floor system of the building or bridge.

The writer disagrees with these statements and looks upon the soil as different from those structural features of a design which are actually designed. For instance, the concrete, steel, or timber members are designed—that is, sizes and materials are chosen from a mathematical operation based upon assumed loads. The soil is not designed, but is found in position, and an attempt must be made to evaluate the characteristics of the material as found (or in some few instances as modified by either physical or chemical means) in order that the static equations of equilibrium can be used. Of course, soils are not uniform, but that in itself cannot be taken as a cause for alarm or an excuse for not attempting to evaluate their characteristics properly. Concrete, timber, and even the ferrous metals are not uniform and are even influenced by a time factor. It has taken a great number of years of experience, trial and error, and experiment to reduce the amount of non-uniformity. In some instances the only advance has been to determine the limits within which characteristics of these materials can be relied upon.

Just as occurs in the testing of concrete, timber, and steel, the testing of soil samples, even with the most modern methods, depends upon the tester as well as upon the type of apparatus. Considerable of the variation reported in

[49] Cons. Engr., New York, N. Y.

[49a] Received by the Secretary November 24, 1941.

soil sampling and soil testing is the human element. There is much less variation between the approximate analyses of soils made by visible methods by two men of long experience with soils than between the results of two separate laboratories operated by different men. One instance, as an illustration, is the comparsion of shear tests on clays, where samples were taken from the identical mass and sent to a number of laboratories in the spring of 1940 by the American Society for Testing Materials.

Personally, the writer would prefer to have the Manual covering pile-driving formulas include a definite formula for granular soils, a definite formula for plastic soils, and a definite formula for such conditions as end-bearing piles in which no lateral restraint or resistance is to be expected.

In all of this, however, it must be emphasized very definitely that the piles in themselves are not a foundation but a means of transferring the load to such layer or layers of soil as can take the imposed load safely within the limited values of settlement. The recommended formula (a simplification of Hiley's formula) is basically of the same type as the "Engineering News" formula. Of course, there is a reduction of the assumed factor of safety from 6 to 3. There is also a more careful evaluation of some of the variables that are substantially constant characteristics for certain cases. However, the writer does not believe that any more accuracy can be obtained from the recommended formula than from the "Enigneering News" formula, if the designer does not have sufficient knowledge to evaluate the factors in each.

The record of foundation failures where piles have been used seldom shows a case in which the cause can be placed upon the use of an improper formula. In almost every instance a study of the failure indicates that the difficulty was with the assumptions made for the application of the formula. There certainly have been failures where no formula was used, where the "Engineering News" formula was used, and where even the latest developments, both theoretical and experimental, have been relied upon. In every case the real trouble was in the assumption made by the designer as to what existed below the surface.

Dynamic load tests are useless in plastic soils. Static load tests will give some dangerous results, even if conducted over a period of time, in plastic soils. Either type of test is satisfactory in granular soils.

The foregoing statements are made without even considering the effect on the results of driving adjacent piles, loading adjacent piles, or even incidental excavation in the vicinity of the test pile.

One criticism of Report *B* can be seriously made as referring to Paragraph *B*–15. The requirement that the allowable load on a pile shall not exceed one half of the load at failure should be defined more carefully, since (in the writer's opinion) failure for piles is a function of settlement and not a physical failure of the pile. Secondly, this paragraph allows the criterion to be the maximum load reached during the test. Such maximum load can be far in excess of one half of the failure load if the test is carried to a point just below failure. In any case, the allowable load should be limited by the desired settlement and should be further limited to only a percentage of the desired settlement of an individual test pile.

It is hoped that the Committee will include in the Manual some recommendations for the lateral resistance of piles and also some, even approximate, method of evaluating such lateral resistance, as well as the pull-out strength of individual piles and of clusters.

The Committee is to be commended for its frank statements in declaring that the problem is far from solved and requires the collection of considerable field data before definite conclusions can be drawn.

Correction: In November, 1941, *Proceedings*, Fig. 3, page 1795, change 'Penetration, in Inches" to "Elevations, in Feet."

AMERICAN SOCIETY OF CIVIL ENGINEERS

Founded November 5, 1852

DISCUSSIONS

SALTS IN IRRIGATION WATER

Discussion

By Messrs. Louis J. Alexander, W. D. Collins, and John H. Bliss

Louis J. Alexander,[11] Assoc. M. Am. Soc. C. E.[11a]—Mr. Hill is to be commended for his paper. He has offered a solution to one of the most perplexing of all geochemical problems; this is, the accurate determination of the source or sources of the resultant mixture of water. Heretofore such solutions were dependent upon the method of trial and error.

Another, and perhaps more important, development in the paper is the method of forecasting what may happen to the soil complex because of the use of water of varying character. If this forecast is applied to one of the local (Southern California) sources, solution of a very controversial problem might be found.

The paper uses the words "character," "type," and "quality" synonymously. Misuse of these terms is common. The writer, however, believes that they should not be so used. "Character" is the "relationship of the ions contained in the solution," one to another and to the whole, and "type" is applicable as a subclassification under "character," whereas "quality" is the "relationship of the concentration of dissolved minerals of other impurities."

The writer has been studying the character, type, and quality of well waters pumped from the Los Angeles Coastal Plain for a number of years.[12] In Southern California consideration of domestic waters must include agricultural and industrial applications. The writer proposes four character types: Two types of carbonate water, a sulfate type, and a chloride type. These four "characters" are defined as follows:

1. Carbonate Waters.—The weak acids exceed the strong acids; alkalies exceed strong acids (no non-carbonate hardness): (CO_3 + H CO_3) exceed (Cl + SO_4); (Na + K) exceed (Cl + SO_4).

Note.—This paper by Raymond A. Hill, M. Am. Soc. C. E., was published in June, 1941, *Proceedings*. Discussion on this paper has appeared in *Proceedings*, as follows: September, 1941, by C. S. Scofield, Esq.; and October, 1941, by Messrs. Herman Stabler, and M. R. Lewis.

[11] Designing Engr., Southern California Water Co., Los Angeles, Calif.

[11a] Received by the Secretary October 14, 1941.

[12] "Character, Quality and Treatment of Well Water Supplies of the Los Angeles Coastal Plain," by Louis J. Alexander, presented at the meeting of the Sanitary Engineering Division of the Society, July 24, 1941, San Diego, Calif.

2. Carbonate Waters.—The weak acids exceed the strong acids; the earths exceed the weak acids (some non-carbonate hardness): (CO_3 + H CO_3) exceed (Cl + SO_4); and (Ca + Mg) exceed (CO_3 + H CO_3).

3. Sulfate Waters.—Strong acids exceed the weak acids and the sulfates exceed chlorides; the earths exceed the weak acids: (Cl + SO_4) exceed (CO_3 + H CO_3); (SO_4) exceeds (Cl); and (Ca + Mg) exceed (CO_3 + H CO_3).

4. Chloride Waters.—Strong acids exceed the weak acids and chloride exceeds sulfate; the earths exceed the weak acids: (Cl + SO_4) exceed (CO_3 + H CO_3); (Cl) exceeds (SO_4); and (Ca + Mg) exceed (CO_3 + H CO_3).

This is a much broader classification than the one proposed by Mr. Hill, but it is believed, for ordinary uses, more satisfactory. This classification is based upon the relative concentrations of the acid groupings rather than upon assumed combinations of various ions and radicals. The reason for this selection is that, in the consideration of the treatment of water, these relationships become important. This is especially true of the waters in sulfate and chloride groups.

A method of study and classification developed by O. A. Stone, M. Am. Soc. C. E., similar to that by Mr. Hill has been used by the writer for some years in the study of localized problems of some complication, but both methods have been found to be somewhat laborious in the study of broad classifications for general work.

Mr. Hill suggests the need for further study. The writer believes that such study should also be enlarged to include waters used for domestic and industrial as well as irrigational uses.

W. D. Collins,[13] Esq. (by letter).[13a]—The new and original method of treating the subject of salts in irrigation waters that is so clearly advanced by the author will undoubtedly prove a useful tool in the hands of those who like to deal with physical and chemical phenomena by means of graphs and equations.

Graphic Representation of Analyses.—Persons not in the habit of using triangular coordinates may be discouraged by the apparent complication of the system proposed by Mr. Hill. The system of graphic representation of water analyses used mainly by the U. S. Geological Survey[14] is based on analyses in terms of equivalents per million, as is the author's system. Figs. 4 to 9 will afford a basis for judging Mr. Hill's method. It seems to some observers that the relative concentrations of the different ions in a water and the chemical characteristics of different waters are a little more easily seen in Figs. 6 and 8. The properties Z_1, Z_2, Z_3, and Z_4 of Mr. Hill's system can be read from the Survey diagrams. In equivalents per million the Z_1 term is the total quantity of sodium when this is less than the chloride, or the total quantity of chloride if the chloride is less than the sodium. The Z_4 property is the total quantity of bicarbonate and sulfate if this is less than the total of the calcium and magnesium, or the total of the calcium and magnesium if this is the smaller. The

[13] Chf., Div. of Quality of Water, U. S. Geological Survey, Washington, D. C.

[13a] Received by the Secretary October 15, 1941.

[14] "Graphic Representation of Water Analyses," by W. D. Collins, *Industrial and Engineering Chemistry*, Vol. 15, 1923, p. 394; also "Notes on Practical Water Analysis," by W. D. Collins, *Water Supply Paper No. 596-H*, U. S. Geological Survey, 1928.

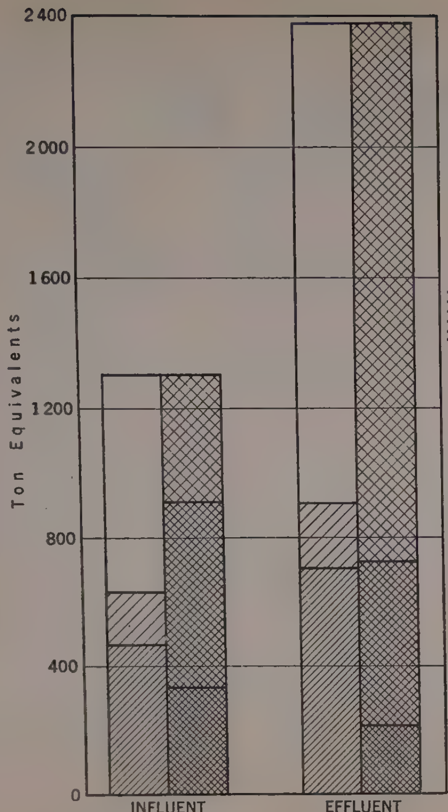


FIG. 4.—INFLUENT AND EFFLUENT WATERS FROM AN IRRIGATED AREA, IN TON EQUIVALENTS

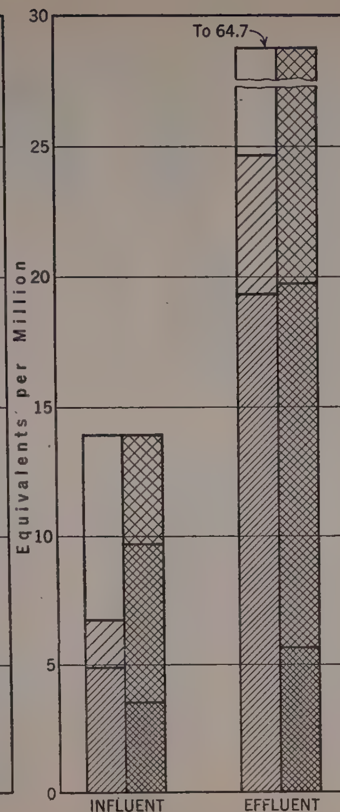


FIG. 5.—INFLUENT AND EFFLUENT WATERS FROM AN IRRIGATED AREA, IN EQUIVALENTS PER MILLION

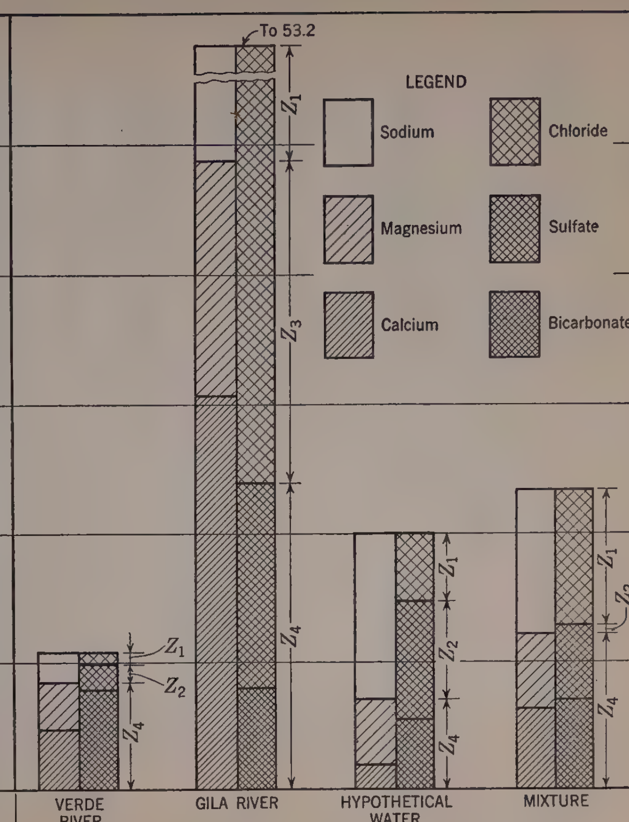


FIG. 6.—RIVER WATERS AND A MIXTURE OF RIVER WATERS, IN EQUIVALENTS PER MILLION

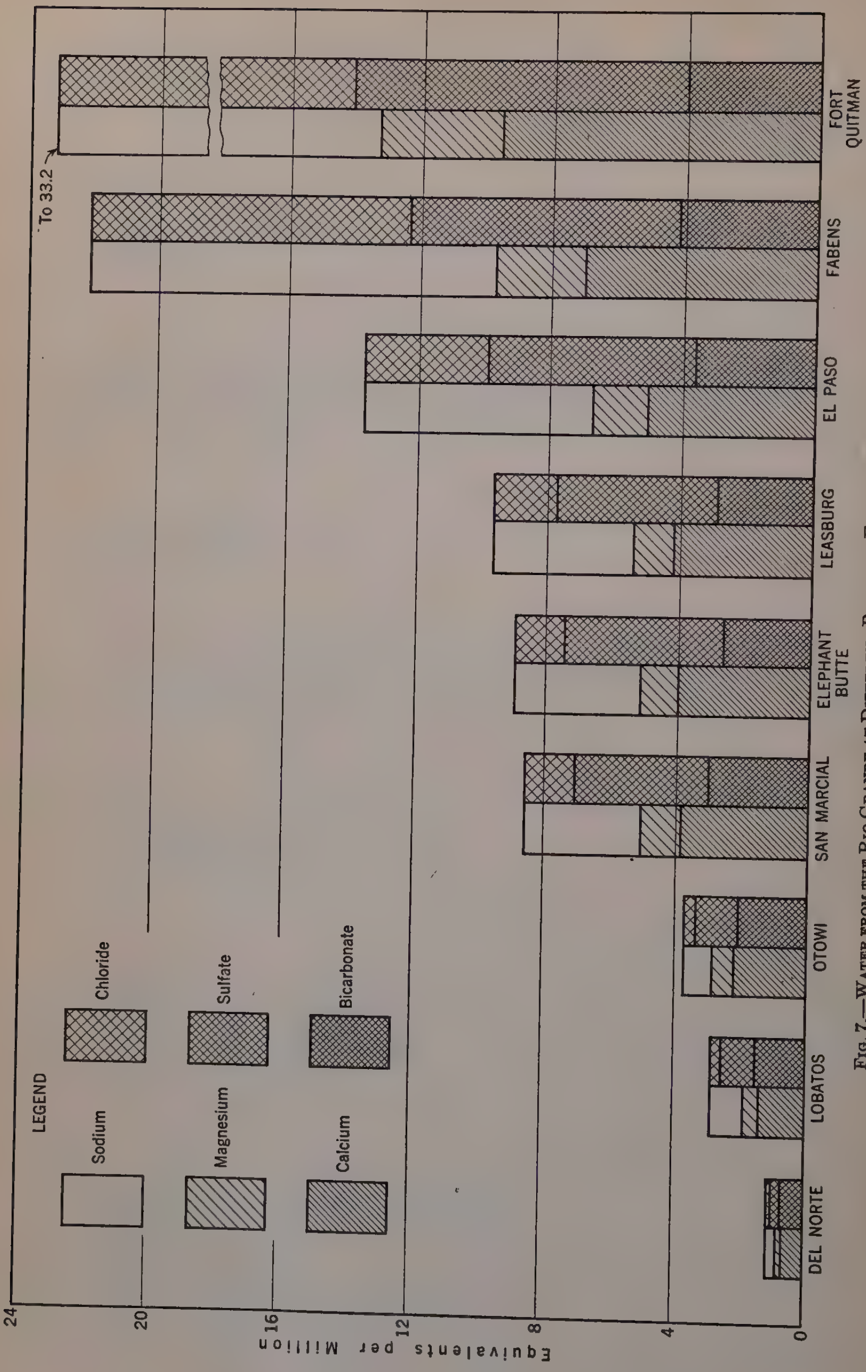


FIG. 7.—WATER FROM THE RIO GRANDE AT DIFFERENT POINTS, IN EQUIVALENTS PER MILLION

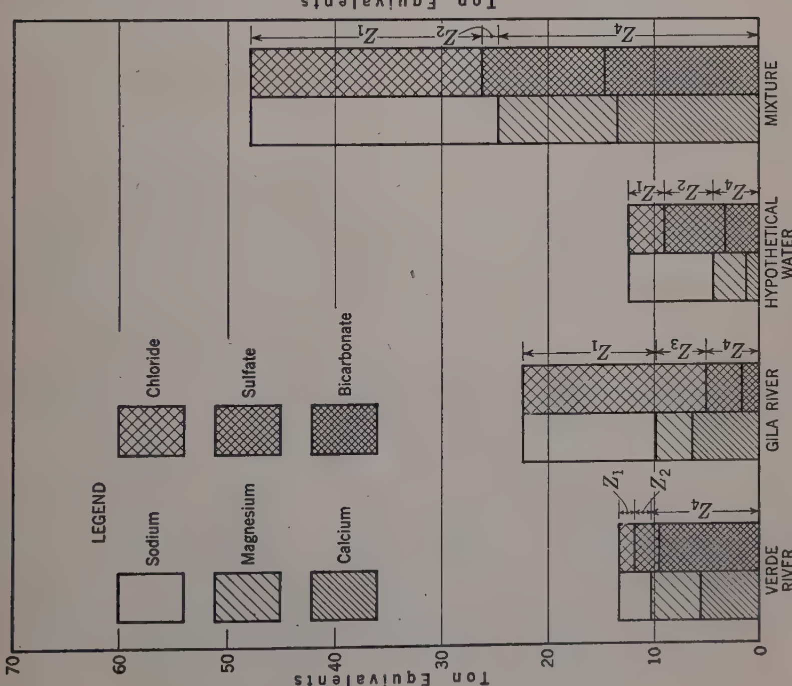


FIG. 8.—RIVER WATERS AND A MIXTURE OF RIVER WATERS, IN TON EQUIVALENTS

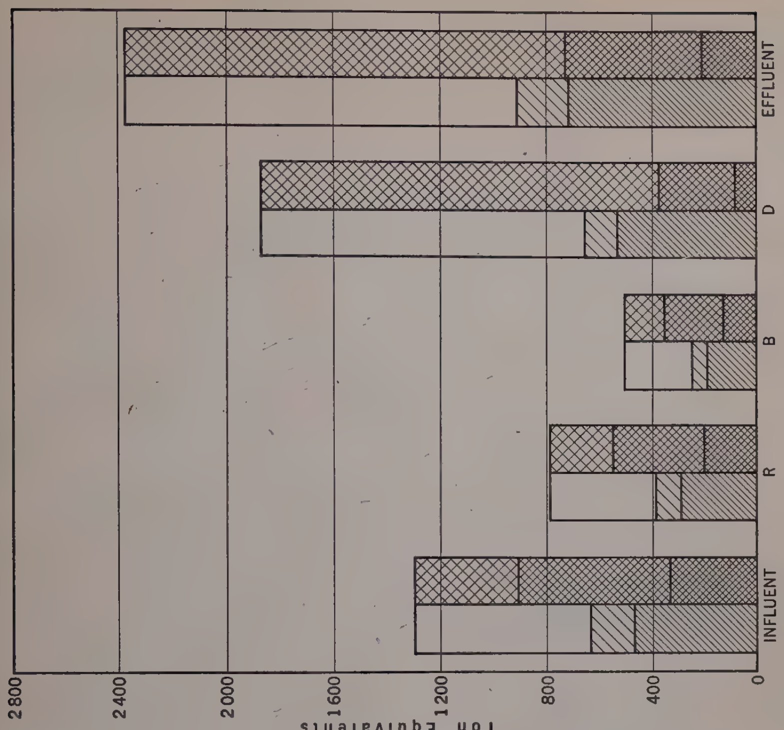


FIG. 9.—SALTS IN IRRIGATION WATERS, IN TON EQUIVALENTS

remainder of the Survey diagram between the Z_1 and the Z_2 parts is either Z_2 or Z_3 according to whether the chloride is less than, or greater than, the sodium. These measurements give the different properties in equivalents per million, which can be converted promptly to percentages by dividing by the total height of the diagram. The Z_1, Z_2, Z_3, and Z_4 properties are indicated in Figs. 6 and 8 which show the analyses of Fig. 1 and Tables 1 and 2.

It is customary to plot Geological Survey analyses in terms of equivalents per million, with the height indicating the concentration. When it is desired to emphasize the percentage composition this can be done by plotting all of the analyses to the same height and representing the concentration of total salts by the width of the diagram.[15]

Mixtures of Different Waters.—The fact that water *M* is a mixture of the other three (which is shown so clearly by the graphs in Table 2) would not be apparent from the analytical results tabulated in equivalents per million or shown in Geological Survey graphs. Granted that the one water is a mixture of the other three, the average person could probably compute the relative quantities of the component waters more quickly by the use of any three of the equations that may be written from the analytical results in the table.

Changes in Quality.—The analyses shown in Table 3 when plotted according to the Geological Survey system (Fig. 7) seem likely to give to the casual observer a clearer picture of the changes in character of the water along the river. Like Table 3, the Survey plot shows an increasing concentration of soluble salts from Del Norte to Fort Quitman. The water at Del Norte, of course, is excellent for all uses. Its total quantity of dissolved mineral matter is not large and the percentage of sodium is small. Increasing quantities of all the soluble salts appear at Lobatos and Otowi without much change in the chemical character of the waters, although the increase is more in calcium and magnesium and bicarbonate than in the other ingredients. From Otowi to San Marcial the water changes in chemical character. As far as Otowi the bicarbonate is the predominating anion and the calcium and magnesium together make up a very large percentage of the cations. To this point the water of the river is a typical calcium-bicarbonate water. The increase in bicarbonate from Otowi to San Marcial is insignificant. The largest relative increase is in sodium and sulfate although the chloride and calcium increase appreciably. The general character of the water remains practically unchanged from San Marcial through Leasburg. At El Paso, Fabens, and Fort Quitman, the sodium, sulfate, and chloride have increased greatly and the calcium has increased moderately, whereas the bicarbonate remains at about the same level.

The changes noted are almost universally characteristic of waters in irrigated areas. When water such as that at Otowi is evaporated to dryness and the residue is treated with distilled water, the calcium carbonate goes into solution slowly and in small quantities, whereas the sodium, magnesium, and calcium chlorides and sulfates go into solution rapidly. The calcium sulfate is not so soluble as the other salts and appears to have more of a tendency to be

[15] "The Significance of Geologic Conditions in Naval Petroleum Reserve No. 3, Wyo.," by W. T. Thom, Jr., and E. M. Spieker, *Professional Paper No. 163*, U. S. Geological Survey, Washington, D. C. (see section on the waters of the Salt Creek-Teapot Dome uplift, by Herman Stabler).

left behind in the land. These considerations appear more easily evident to the average observer from the Geological Survey graphs although they are all shown in Table 3.

Valley Salt Balance.—In order to consider the valley salt balance shown in Table 4, the analyses of influent and effluent waters have been plotted according to the Geological Survey system in two units (Figs. 4 and 5). They are shown first in ton equivalents, as in Table 4. To obtain a better picture of the actual composition of the waters the two analyses are also plotted in equivalents per million. A glance at the diagrams in ton equivalents (Fig. 4) might suggest that the outgoing water was lower in bicarbonate than the incoming water. The diagrams in equivalents per million (Fig. 5) show that the effluent water is more concentrated than the influent water in every constituent. The fact that the effluent shows fewer tons of bicarbonate and sulfate than the influent is due in part to the fact that the volume of the effluent is only 27,000 acre-ft to 69,000 acre-ft for the influent.

The discussion by Mr. Hill centering around the imaginary water called "*A*" is ingenious and interesting and leads to practical conclusions that seem wholly sound. It would appear that substantially the same conclusions might be reached by direct consideration of tangible quantities without resort to an imaginary water.

From the table of analyses or from the Survey diagrams (Figs. 4 and 5) it can be seen readily that the effluent salts (or water) from the plot under consideration have a different character from the influent salts and water. The difference is not one of simple concentration as by evaporation of water, nor could it be brought about by the normal phenomena common in irrigated regions of precipitation of calcium carbonate or calcium sulfate. Besides the relative increase of sodium as compared to calcium, and of chloride as compared to bicarbonate, the diagrams show clearly, in equivalents per million, a shift of the sodium-magnesium boundary in relation to the chloride-sulfate boundary. In terms of hypothetical combinations the influent water carries sodium sulfate and no chloride of magnesium or calcium and the effluent water carries no sodium sulfate, but does have some magnesium chloride. In Mr. Hill's terms the diagrams of influent water show the Z_2 property and the effluent diagrams show Z_3. The diagrams do not show any imaginary water or any loose unattached or unbalanced ions such as can be made to appear from the analytical results in Table 4.

TABLE 7.—ANALYSES OF INFLUENT AND EFFLUENT WATER FOR AN IRRIGATED VALLEY OVER A PERIOD OF ONE YEAR; WITH CERTAIN COMPUTATIONS FROM THE ANALYSES (RESULTS IN TON EQUIVALENTS)

Ion	I	R	D	E' $(=I+D)$	E $(=E'-R)$	G $D-R$ $E-I$
Ca...	465	122	366	831	709	244
Mg...	166	60	89	255	195	29
Na...	664	0	806	1,470	1,470	806
Sum..	1,295	182	1,261	2,556	2,374	1,079
HCO_3	332	122	0	332	210	−122
SO_4..	576	60	0	576	516	− 60
Cl....	387	0	1,261	1,648	1,648	1,261
Sum..	1,295	182	1,261	2,556	2,374	1,079

It is recognized that an actual irrigated plot of 15,000 acres does not act like a glass vessel or like a bed of pure silica sand. The action with respect to applied water varies from spot to spot in the area. The analyses *I* and *E* in Table 4 give only an over-all picture of the relation of incoming and outgoing water and salts at a given time or over a given period. Assuming that the analyses are truly representative of actual conditions it appears that the influent water brought into the area more bicarbonate and sulfate than the effluent water removed, and that the effluent water took away more chloride than was brought in.

The simplest over-all solution of the problem would be to assume that some of the incoming calcium or magnesium bicarbonate and sulfate remained in the soil and that the outgoing water removed some calcium or magnesium chloride from the area. This can be set up as in Table 7 which shows, in ton equivalents:

I, the constituents of the influent water;

R, the tons of calcium bicarbonate and magnesium sulfate left in the area, out of the amount brought in by the influent water;

D, the tons of different constituents displaced from the store of soluble salts in the area at the beginning of the year;

$E' = I + D$, the sum of the influent salts and the salts displaced from storage in the ground (this might be called the over-all total of salts moved into and in the area);

$E = E' - R$, the difference between the tons of salts moved into and in the area and the tons of influent salts left in the ground; and

$G = D - R = E - I$, the apparent over-all removal of salts from the area.

It is obvious that any number of solutions can be made by adding equal quantities to corresponding items in Cols. *R* and *D*. Quantity *E* will remain the same. The top limit will occur when *R* is the same as *I*. That will mean that none of the influent salt for the year was carried out of the area and the effluent salt was all from the store of soluble salts in the area.

TABLE 8.—RESIDUAL (*R*) AND DISPLACED (*D*) SALTS, AND APPARENT GAIN (*G*) IN AN IRRIGATED AREA DURING A PERIOD OF ONE YEAR

(Results in Ton Equivalents)

Ion	Assumption 1[a]			Assumption 2[b]			Assumption 3[c]			Assumption 4[d]		
	R	*D*	*G*	*R*	*D*	*G*	*R*	*D*	*G*	*R*	*D*	*G*
Ca	122	366	244	214	458	244	283	527	244	465	709	244
Mg	60	89	29	101	130	29	101	130	29	166	195	29
Na	0	806	806	230	1,036	806	404	1,210	806	664	1,470	806
Sum	182	1,261	1,079	545	1,624	1,079	788	1,867	1,079	1,295	2,374	1,079
HCO_3	122	0	−122	214	92	−122	202	80	−122	332	210	−122
SO_4	60	0	−60	276	216	−60	350	290	−60	576	516	−60
Cl	0	1,261	1,261	55	1,316	1,261	236	1,497	1,261	387	1,648	1,261
Sum	182	1,261	1,079	545	1,624	1,079	788	1,867	1,079	1,295	2,374	1,079

[a] No residual salts except the minimum bicarbonate and sulfate and no extra displaced salts except the equivalent calcium and magnesium chlorides. [b] The solution by the use of the imaginary water and diagrams of Mr. Hill's paper. [c] The salts in 27,000 acre-ft of influent water pass on through; the salts in 42,000 acre-ft of the influent remain in the area. [d] None of the influent salts leave the area; all the effluent salts come from the stored salts.

Table 8 shows values of R and D for different assumptions. Fig. 9 shows, in ton equivalents, the influent salts in 69,000 acre-ft of water, the residual salts R from 42,000 acre-ft of the influent water, the salts carried through the area by 27,000 acre-ft of the influent water B, the salts taken up from the store in the area D, and the effluent which is the sum of B and D (see Assumption 3, Table 8).

The consideration of the relations between influent and effluent waters, whether by means of Mr. Hill's charts, by means of computation from the analyses, or by the Survey diagrams, points to the possible danger of accepting a false sense of security with reference to the salinity of lands in an irrigated area merely because the over-all salt balance seems to be favorable. The results show the need for continuous study of the character of the waters in contact with the soil as well as the influent and effluent waters for individual plots if salt damage to the land is to be avoided.

JOHN H. BLISS,[16] ASSOC. M. AM. SOC. C. E.[16a]—An interesting and ingenious method of plotting and classifying irrigation waters has been advanced by Mr. Hill. His geochemical chart is an excellent tool for portraying the relative amounts and characteristics of dissolved solids and for comparing differences in water supplies. It is also an aid in solving certain problems such as valley salt balance, although, in this connection, it does not appear to be quite so successful. The difficulty is not in the method but in the inadequacy of the basic data.

Because of the number of dissolved solids and the complexity of the effect of their active constituents upon crops and soils, it is obviously difficult, in a simple graph, to tell the entire story of a water's character and suitability for irrigation purposes. The problem is a complex one and the author is to be congratulated for his effort to simplify it and to standardize the method of reporting the results of water analyses.

By combining the strong sulfate ion with the weak carbonates and bicarbonates, rather than the chlorides and nitrates, Mr. Hill has departed somewhat from the Palmer method[2] of classification. By so doing he has tended to overlook the geologic significance of the Palmer classification in order to portray better the irrigation significance of natural waters.

The writer is somewhat familiar with the Rio Grande, one of the illustrative examples used by the author, and is particularly interested in his discussion of valley salt balance (upper El Paso Valley). Arriving at an indeterminate mathematical solution, Mr. Hill has solved the problem by reasonable deduction and has obtained an approximate answer to the true salt debits and credits in the area considered.

It is obvious that a permanent agriculture depends upon the maintenance of a salt balance in the root zone of cropped areas—that is, that as much of the undesirable dissolved constituents are removed from the root zone as are carried to it in the irrigation supply. The author has demonstrated that a

[16] Engr., State Engr's. Office, Santa Fe, N. Mex.

[16a] Received by the Secretary November 6, 1941.

[2] "The Geochemical Interpretation of Water Analyses," by Chase Palmer, *Bulletin No. 479*, U. S. Geological Survey, 1911.

balance between influent and effluent salts in an irrigated valley does not necessarily mean a favorable balance to the growing crops. The writer agrees, but rather questions the inference that, because unfavorable residual salts are being left behind somewhere in the soils of an area, the entire valley is affected adversely. In the illustrative case it would seem that many if not most of the residual salts were left in those areas of relatively impervious soils whose useful life as irrigated lands are automatically limited by their "tightness." In the remaining more porous soils of the valley, any residual accumulation may be removed easily from time to time by flushing the soil with excess quantities of water.

Classifications of water according to their suitability for irrigation purposes have been advanced for certain limited areas. Realizing the pitfalls that surround any attempt at broad classification of waters, the writer feels that there is much work to be done along this line. C. S. Scofield[8] has presented a table of classes of irrigation water, which (he warns) are adopted for a definite irrigation region. The table sets up five classes of water from "excellent" to "unsuitable" and gives permissible limits of concentration of those constituents likely to be injurious to growing crops. For example, Class 5, or "unsuitable" irrigation water, is described in part as carrying more than 2,100 ppm of dissolved solids, more than 80% (of total positive ions) sodium, and more than 20 milligram equivalents each of sulfates and chlorides.

The following examples may be of interest and are included herein because the writer believes that they demonstrate the need for additional information on the effects of irrigation waters upon soils and crops.

The Pecos River in New Mexico and Texas carries one of the highest concentrations of dissolved solids of any western stream used for irrigation. For the three years 1938–1940, the water supply for the Carlsbad (N. Mex.) project, as measured on the Pecos River near the Artesia (N. Mex.) gaging station by the U. S. Geological Survey,[17] averaged 3,120 ppm of total dissolved solids, 37% sodium, and 28.8 and 18.5 milligram equivalents, respectively, of sulfates and chlorides. Water of this type and approximate average concentration has been applied on the project lands for almost fifty years with little apparent adverse effect except in those areas where drainage is difficult.

Texas projects, now combined in a single district under the Red Bluff reservoir, receive a much more unsuitable water supply than the Carlsbad project upstream. The quality of the district's irrigation supply as measured by the U. S. Geological Survey at the Orla (Tex.) gaging station below the reservoir, for the 1938–1940 period, averaged 4,520 ppm of total dissolved solids, 44% sodium, and 33.2% each of sulfates and chlorides. On the Pecos River at Red Bluff (N. Mex.) station above the reservoir, the sodium content for the same period averaged about 52%, a more representative value than that at Orla because the Orla data are affected by the holdover storage of the relatively good waters caught in the reservoir during the flood of June, 1937. Water of about the Red Bluff type and concentration has been used on Texas lands for

[8] "The Salinity of Irrigation Water," by C. S. Scofield, The Smithsonian Rept. for 1935, pp. 275–287.

[17] "Quality of Water," by C. S. Howard, W. F. White, Jr., and W. W. Hastings, U. S. Geological Survey—a section of the report, "Pecos River Joint Investigation," National Resources Planning Board, 1941.

many years. Considerable areas of heavy soils, particularly those in the flood channel along the river, have been "alkalied" and abandoned. However, large areas where natural or artificial drainage has been effective have been farmed successfully for many years with little apparent adverse effect.

The classification of these two waters according to the methods of Messrs. Palmer and Hill is shown in Table 9. It will be noted that the water at Red Bluff falls in Mr. Hill's Type VII, which he aptly calls "irrigation sewage."

TABLE 9.—CLASSIFICATION OF PECOS RIVER WATERS

Station	Palmer Classification				Hill classification (type)
	Class	Primary salinity	Secondary salinity	Secondary alkalinity	
Artesia...	3	37%	58%	5%	*b*I*a*
Red Bluff	3	52%	45%	3%	*d*VII*d*

Both waters, except as to sodium percentage, fall in Mr. Scofield's Class 5 or "unsuitable" irrigation water (assuming the Scofield classification to be applicable to this area). A favorable predominance of the earth bases over sodium and potassium exists in the Artesia water, but this advantage is lost at Red Bluff where sodium and potassium constitute more than 50% of the positive ions. There is insufficient evidence at this time to show what effect "base exchange" may have made upon the soils in either area.

How long successful irrigation agriculture can endure, particularly in the Red Bluff district, under the continued use of such highly concentrated, inferior quality water remains to be seen. It would appear, however, that even poor quality saline water may be used more or less successfully for irrigation purposes for considerable periods of time if the excess salts are flushed away from time to time and thus prevented from accumulating in the root zone of growing crops. There is a definite need for additional study of this complex problem.

AMERICAN SOCIETY OF CIVIL ENGINEERS

Founded November 5, 1852

DISCUSSIONS

THE SURVEYOR AND THE LAW

Discussion

By D. D. Haines, Assoc. M. Am. Soc. C. E.

D. D. Haines,[70] Assoc. M. Am. Soc. C. E.[70a]—In his "Introduction" Professor Holt makes a very good point when he states that a surveyor "must be ably advised on any given point concerning the law in the jurisdiction in which he is concerned." The mere fact that the law of one state provides that certain procedures must be followed in that state under certain conditions does not necessarily mean that the same procedure should be followed in another state. Statements of good law in one state must never be made to apply in another. A certain amount of local research on the subject must always be made. In laws relating to boundaries each state is an entity unto itself.

It is a well-known fact that not all states require that a surveyor be licensed, or registered, before he can do boundary surveying. However, if a surveyor is registered in a particular state, it is assumed that he is a "qualified" land surveyor; such qualification means that the surveyor has at least the minimum ability to exercise the necessary "ordinary prudence." Students invariably ask, "What is ordinary prudence?" This is a very natural question and one that defies a definite answer. Professor Holt has given the basis for the answer to such a question. Records must be pursued, boundaries examined, occupancy determined, and any other information carefully weighed; and furthermore the surveyor must tell his client just what the facts are as he sees them. The responsibility of a surveyor to his client is the same whether he is a registered or an unregistered practitioner. Should a surveyor fail to exercise "ordinary prudence," it is well known that he can be held liable for damages. To eliminate errors and secure the necessary precision in boundary surveying is not peculiar to that branch of surveying, nor to surveying in general, but to engineering as a whole. Professor Holt mentions the custom of giving a "certificate of accuracy" as a usual way of emphasizing accuracy. The writer would like to have Professor Holt enlarge on this subject.

Note.—This paper by A. H. Holt, M. Am. Soc. C. E., was published in May, 1941, *Proceedings*. Discussion on this paper has appeared in *Proceedings*, as follows: October, 1941, by Messrs. E. F. Chandler, Harry Rubey, William H. Richards, Jr., and C. B. Humphrey.

[70] Asst. Prof. of Civ. Eng., Univ. of Kansas, Lawrence, Kans.

[70a] Received by the Secretary November 21, 1941.

The writer is frequently asked by students if a surveyor is actually a trespasser. The reply always has been that he is unless he has permission, or is exercising an official duty, and even then the surveyor is liable for actual damages as well as for any unnecessary damage he does. The author has made this point very clear. In the majority of cases, a surveyor can secure the permission of the owner to enter upon his land merely by explaining the need for such entry and promising to exercise due care. The surveyor should always keep in mind that he cannot force his way on to private property unless he has official authority, or unless a state statute definitely provides for such entry, and that in the majority of cases it is far better not to use authority unless absolutely necessary.

It is well that this paper states that any survey will "hold" and become a "legal" survey when: (1) The results are accepted by those affected; or (2) the court rules that the survey properly settles the problem. Therefore, private surveyors and official surveyors have equal authority to "establish boundaries." Not enough laymen understand this fact.

Boundary disputes can be settled without court action. The realization of this fact is frequently startling to the young engineer or surveyor. Agreement may be reached between adjacent landowners when not in the presence of a surveyor, but since a surveyor is generally a part of such discussions he frequently can serve as an arbitrator. The surveyor is a better arbitrator if he understands the law thoroughly and if he has collected and weighed all the facts. Any award he makes will be considered valid, and any boundary fixed by agreement remains fixed regardless of mistakes of judgment or measurements. Although agreements to fix boundaries need not be in writing, it is better practice to make them of permanent record. One point cannot be overemphasized with respect to boundaries established by agreement and that is that the correct boundary line still has its place and importance, in that it (and not the agreed line) must be used as a starting point in the location of other lines and monuments.

The writer would like to have Professor Holt enlarge on the statement: "Such an agreement to fix a boundary line is void if the owners know or if one of them knows that the agreed line is not the true line" (see heading "Arbitration").

Professor Holt is quite right when he states that attorneys of general practice are not always entirely familiar with matters of boundaries. In fact, he could go further than that and state that they are seldom more than moderately familiar with such matters. Along this same line it should be noted that surveyors should not hesitate to consult lawyers as to the correct procedure to follow under certain circumstances. There must be a certain amount of "give-and-take" among the two professions to solve problems to their mutual advantage.

The discussion on the admissibility of plats and field notes is very interesting. The author has given a good bibliography for reference. It seems that field notes and plats from any source, so long as they are properly identified and authenticated, may, under the usual circumstance, be admitted as testimony.

The writer is glad that Professor Holt has seen fit to express his opinion to

the effect that the most useful form in which a survey can be submitted to a client is a plat, thus leaving the survey notes in their original form with the original surveyor. Of course, the plat must be adequate and must give all the final data pertinent to the survey that the surveyor's client requested. The author's point (see heading "Surveyor's Report to His Client") that "graphical descriptions are often not only much more convenient than verbal ones, but much less likely to be misunderstood," is a good one and should cause more and more plats to be made and appended to legal description.

The entire paper is very interesting and worth while. Although most of the points discussed are problems that have been present for some time, they are constantly recurring, and are of sufficient importance to make it worth while to have them so ably reviewed.

PROCEEDINGS

American Society of Civil Engineers

DECEMBER
1941

VOLUME 67 NO. 10

PROCEEDINGS
OF THE AMERICAN SOCIETY OF CIVIL ENGINEERS

SYDNEY WILMOT
Manager of Publications

HAROLD T. LARSEN
Editor of Proceedings

COMMITTEE ON PUBLICATIONS

HAROLD M. LEWIS, *Chairman*

C. G. DUNNELLS
CHARLES T. LEEDS
LAZARUS WHITE
R. B. WILEY

AMERICAN SOCIETY OF CIVIL ENGINEERS

COMING MEETINGS

Board of Direction Meetings

January 19-20, 1942:

A Quarterly Meeting will be held in New York, N. Y.

April 20-21, 1942:

A Quarterly Meeting will be held in New Orleans, La.

ANNUAL MEETING

NEW YORK, N. Y.

January 21, 22, 23, 1942

SPRING MEETING

NEW ORLEANS, LA.

April 22, 23, 24, 1942

The Reading Room of the Society is open from 9:00 A.M. to 5:00 P.M. every day, except Saturdays when it is closed at 12:00 noon. It is closed all day on Sundays and holidays.

Members, particularly those from out of town, are cordially invited to use this room on their visits to New York, to have their mail addressed there, and to utilize it as a place for meeting others. There is an ample file of current periodicals, the latest technical books, and the room is well supplied with writing tables.